CHAPTER

1 Introduction

Under the term "Irrigation" as applied to the agriculture, is included all of the operation, or practices in artificially applying water to the land for the production of crop during base period of crop. Or irrigation may also defined as "the artificial process of supplying water for the growth of crops as per crop requirement in the absence of sufficient precipitation".

Ordinarily water is supplied to the crop land by nature through rain but generally it is not sufficient for the proper growth of the plants. Different types of plants require different quantities of water at different times, till they grow up completely. In order to achieve this objective of irrigation, an irrigation system is required to be developed, which involves planning, design, construction, operation and maintenance of various irrigation works viz. a sources of supply, a different system for carrying water from the source to the agricultural land and its application on the land, and various other associated works.

Irrigation involves artificially providing water to crops. This technique is used in farming to enable plants to grow when there is not enough rain, particularly in arid areas. It is also used in less arid regions to provide plants with the water they need when seed setting. About 66% of the world's water catchment is used in farming, which continues to make increasing use of irrigation. But in most irrigation systems 50 to 60% of the water used does not benefit the plants. It is therefore necessary to set up more carefully designed irrigation schemes that ensure optimum agricultural production while preserving this resource.

1.1 Need and Scope of Irrigation and Drainage in Nepal

Nepal is agricultural country where most of the people are dependent in agriculture. But the rainfall, climate, topography and vegetation vary from place to place. The amount and duration of rainfall vary considerably. Even at particular place, the rainfall is highly erratic and irregular, as it occurs only during a few particular months of the year. Crops cannot be grown properly over the entire land without providing artificial irrigation in fields.

If the normal rain at any place is adequate to meet the total water requirements of the crops grown and the time of interval of the rainfall is such that the water is available whenever the plants need it, then irrigation is not required. Such ideal condition exists only for some regions of Nepal where the normal rainfall is sufficient to fulfill the water requirements of the crops grown. However for most of the regions of Nepal crop production is not possible without irrigation. Thus the factors which necessitate irrigation are indicated below.

(i) Inadequate rainfall

When the rainfall at a place is inadequate to meet the crop water requirements, then it would be necessary to use irrigation. In such cases water may be conveyed from the places where it is available in abundance to the place of deficiency.

(ii) Uneven distribution of rainfall

The total rainfall in a region may be adequate but it may be unevenly distributed over time as well as space. The entire rainfall for any region may occurs only during some period of the year and for the rest of the period there may be no rain. Further in some part of the region there may be excessive rain and remaining part may have little or almost no rain. The rainfall in Nepal has such characteristics. The entire rainfall is received from the monsoon during the four months from June to September and the rainfall during winter is scanty. With such uneven distribution of rainfall, it would be necessary to use irrigation.

(iii) Growing crops round a year

The rainfall in a region may be sufficient to grow only one crop in a year for which no irrigation may be required, for example rice in monsoon. However, in the same region if more numbers of crops are to be grown during the same year it would be possible only if the irrigation facilities are available. Moreover for some crops such as sugarcane have much longer period of maturity for which irrigation is invariably needed.

(iv) Growing superior crops

Certain inferior or low-priced crops require less water and hence for growing such crops only the rain water may be sufficient and no irrigation is therefore needed. However there are several superior or high- priced crops which need frequent application of large quantity of water and for growing such crops, irrigation is necessary.

There is a huge scope of irrigation development in Terai as well as in Hills of Nepal. In Terai (Plain areas) a lot of agricultural areas are available without irrigation facility. These plains have high fertility but in the absence of irrigation facilities they are limited to rain-fed cropping only. In Terai there is a future scope of developing large irrigation systems with barrages, small dams and reservoirs to produce a variety of crops in large quantity.

In hills there are small commanded areas which need small irrigation systems. In hills a lot of Farmer Managed Irrigation Systems (FMIS) are available but they are not designed properly by engineers. Farmers are facing difficulty in operation and maintenance of these systems. Hence there is a scope for improving these small FMIS and construction of new irrigation systems in hills of Nepal.

Irrigation and drainage are two face of the same coin. Surface irrigation is a boon only if it is practiced with great care. Due to steep topography, mountains and hills are less susceptible to water logging, hence less need of drainage. In hills and mountains due to presence of sufficient natural drainage there is less requirement of drainage. Remodeling of exiting natural drainage is sufficient to drain off the excess water. But in case of plain areas during monsoon season, there is great chance of water logging. To remove the excess water, there is necessary to introduce the surface and subsurface drainage. By providing drainage facilities in Terai, optimum crop production can be assured.

1.2 Challenges and Opportunities of Irrigation Development in Nepal

The irrigation development will lead to economic growth of the country by the increase in the production of crop, intense industrialization, employment generation and so on. The irrigation development can replace the purchase of food from foreign. It can also reduce the famine problem.

The main challenges in irrigation development in Nepal have been identified as follows:

- The topography and fragile geology create more challenges to development of irrigation project.
- Monsoon and/or rainfall pattern are highly variable in respect of time and place.

- In hills where more head is available there is less agricultural fields and in plains where more agricultural fields are available there is less head.
- Isolated communities have out of necessity concentrated on growing subsistence food crops. Due to difficult remote area, isolated farmers communities are not encouraged to adopt new cropping pattern for want of market access.
- Due to poor agricultural support services improved seed varieties and agricultural techniques are often not available to the farmers.
- Due to deforestation, rainfall runoff is more rapid, increasing the frequency of severs floods.
- Landslide and erosion causing damage to irrigation structure (particularly intakes) and clogging canals.
- Conflicts between water users and political interferences.
- Misconceptions and lack of coordination between developer agencies.
- Problem in collection of Irrigation Service Fees.

There are several challenges as above mentioned however there are good opportunities for irrigation development in Nepal. For irrigation development we need sufficient water source, croplands, suitable crops, suitable climate, skilled human resources, construction materials and technology. These all are more or less available in Nepal. For the source of irrigation water in Nepal different rivers, streams, lakes, springs and ground water can be used. Agricultural and crop lands are abundantly available in all parts of the country. Different types of crops such as rice, wheat, maize, tobacco, sugarcane, millet including vegetables are suitably grown in the country. The climate of Nepal is also favorable for growing different variety of crops and fruits. At present we have sufficient national experts in different areas for designing and construction of irrigation projects. Basic construction materials such as cement, sand, brick, boulder and steel are also available in Nepal. Fast growing technology can also be used to develop the irrigation projects.

1.3 Status of Irrigation Development in Nepal

Out of the total 14.718 million hectare area of the country only 2.641 million hectare area is arable and 1.766 million hectare land is irrigable. 76% of potential irrigable area lies in the Terai region of Nepal.

The history of irrigation in Nepal before 1922 shows that irrigation were developed, operated and maintained by farmers called Farmers Managed Irrigation System (FMIS). From 1922 to 1957, Government made little effort to develop irrigation infrastructures in Nepal. Chandra Nahar, Juddha Nahar, Jagadispur (Banganga), Phewa Dam are few examples of the projects developed during that period. However, irrigation infrastructure development has got high priority since 1957, the milestone of the beginning of periodic plan in Nepal.

The minor irrigation program was introduced in the second three-year development plan (1962-65) to provide low-cost-irrigation facilities to farmers within a short period of time. The program included the construction of small wells, irrigation tanks, reservoirs, pumps (lift) and other low cost irrigation facilities. Although it was planned to provide irrigation facilities to 4,455 hectares by the end of the Plan period under this program, the actual achievement was insignificant. The Third Plan Period (1966-70) saw the countrywide implementation of the minor irrigation program with the emphasis on the participation of the beneficiaries. The government investment in irrigation development – especially in the large-scale irrigation systems in the Terai increased tremendously from 1970 onwards. This was due to the increase in the borrowing of international capital in the form of loans and grants for the country's overall economic development. This is clearly reflected in the surge of irrigation development targets in the subsequent five-year development plans- from the Fourth Plan (1970-75) onwards.

Until the middle of 1980s, irrigation development by the government focused largely on the construction of physical infrastructure of canals and structures, and very little attention was given to the effective management of the completed systems. Attention began to be paid to the improved management of government-operated irrigation systems from 1985 onwards. This is reflected in the implementation of a number of management-oriented projects in 1985-89: the USAID-funded Irrigation Management Transfer Project (IMTP) in 1985, the Irrigation Line of Credit (ILC) in 1988 financed by the World Bank, the Irrigation Sector Project (ISP) in 1988 financed by the ADB, and the Irrigation Sector Support Project (ISSP) in 1989 under the co-financing of the UNDP, the World Bank and the Asian Development Bank (ADB). All these projects have specifically emphasized the participatory approach to irrigation development and management of irrigation facilities. Further, following the introduction of the Basic Needs Program (BNP) in 1987, the working Policy on Irrigation Development for the fulfillment of Basic Needs' was formulated in the early 1989.

This was immediately followed by the promulgation of the Irrigation Regulations (IR) in April 1989. These Regulations placed emphasis on the greater collaboration with water users in all phases of irrigation projects – planning, construction, operation and maintenance. The strategy of increasing farmer participation was mainly based on the recognition that government resources alone were inadequate to meet the country's irrigation development objectives and sustain the management of government irrigation systems after their completion. The government expected to increase the rate of irrigation development and develop maximum farmers'/water users' responsibility in the operation and maintenance of completed irrigation systems. The Irrigation Regulations gave water users, for the first time, a legal mandate to form water users' associations in accordance with the 1976 Association Registration Act. It institutionalized the participation of actual water users in irrigation. In 1989, the action plans and policies for the turnover of small irrigation systems and the participatory management of large irrigation systems were formulated.

This was followed by the promulgation of Water Resources Act and Irrigation Policy in 1992 with the clear vision of irrigation development. Later this policy was amended in 1997 and now Irrigation Policy 2004 is in practice. Similarly Irrigation Master Plan 1990, Agriculture Perspective Plan 1995, Water Resources Strategy 2002 and National Water Plan 2005 are other few documents which guide irrigation development in Nepal.

Out of total area 1,47,181 km^2, Nepal has cultivable area of 26,42,000 ha. Out of total cultivable area irrigable land is 17,66,000 ha. At the end of F.Y. 071/72, the area covered by the different irrigation methods are as follows.

Surface irrigation = 779694 ha.

Subsurface irrigation = 391080ha.

Farmers channel = 198140 ha.

Some irrigation systems of Nepal are given in Table 1.1 with their commended area and average design discharge.

Table:1.1 List of irrigation system and their commanded area with discharge (Source: - Department of Irrigation)

S.N	Irrigation System	District	Commanded Area (ha)	Asar-Kartik		Mangsir-Falgun		Chaitra-Jestha		Possible Irrigated area (ha)
				Average Discharge (lps)	Irrigated area (ha)	Average Discharge (lps)	Irrigated Area (ha)	Average Discharge (lps)	Irrigated Area (ha)	
1	Kankai IS	Jhapa	8000	8000	7000	6000	4000	3000	2500	0
2	Sunsari Morang IS	Morang and Sunsari	68000	50000	66000	22000	58000	15000	17700	0
3	Chandra Mohana IS	Sunsari	1800	1650	1500	1500	1000	1100	900	0
4	Chandra Canal IS	Saptari	10500	8000	10000	6000	7000	1500	300	100
5	Koshi West Canal IS	Saptari	11000	9900	11000	9900	9000	9900	5000	7000
6	Kashi Pump Canal IS	Saptari	13000	7000	10000	5000	7000	5000	0	2000
7	Kamala IS	Siraha and Dhanus ha	25000	28800	25000	8000	10000	4000	0	2000
8	Hardinath IS	Dhanus ha	2000	1600	1700	400	800	210	200	0
9	Manushm ara IS	Sarlahi	5200	5800	5000	4300	3000	3200	1500	0
10	Bagmati IS	Sarlahi and Rautaha t	45600	34000	39700	10200	25500	7700	13000	0
11	Jhaanjh IS	Rautaha t	2000	3500	2000	1200	1500	1200	500	500
12	Narayani IS	Bara and Parsa	28700	19200	28000	12000	18000	10000	500	12000
13	Narayani Tubel IS	Bara and Parsa	2800	280	600	280	600	100	300	0
14	Narayani Lift IS	Chitwa n	4700	5500	3700	2000	300	2000	0	3200
15	Khageri IS	Chitwa n	3900	6200	3600	2100	500	1500	500	0
16	Pokhara Jal Upayog IS	Kaski	1030	9000	1000	6000	500	4000	200	0

S.N	Irrigation System	District	Commanded Area (ha)	Asar-Kartik		Mangsir-Falgun		Chaitra-Jestha		Possible Irrigated area (ha)
				Average Discharge (lps)	Irrigated area (ha)	Average Discharge (lps)	Irrigated Area (ha)	Average Discharge (lps)	Irrigated Area (ha)	
17	Bijayapur IS	Kaski	1280	6000	1100	3000	500	2000	500	0
18	Begnas IS	Kaski	580	4500	500	1500	200	1000	100	0
19	Fewa IS	Kaski	330	7000	320	3000	100	1000	20	0
20	Phalebash IS	Parbat	440	700	340	350	130	100	75	0
21	Nepal Gandak West Canal IS	Nawalparasi	10300	8500	10000	4500	8000	4000	1300	2700
22	Bhairahawa Lumbini Underground Water Resource IS	Rupandehi	20309	10500	13500	10000	8000	10000	5000	0
23	Marchwaar lift IS	Rupandehi	3500	5200	3200	4000	2500	3200	0	600
24	Baanganga IS	Kapilbastu	6200	3000	6000	2500	3600	1500	100	0
25	Praganna Canal IS	Banke	5800	25000	5600	15000	5600	6000	600	3500
26	Dunduwa IS	Dang	1250	2046	500	1527	200	300	50	0
27	Chaurjahari IS	Rukum	600	1200	600	500	300	300	0	200
28	Babai IS	Bardiya	13500	18000	11000	6500	6500	3000	1500	1500
29	Rajapur IS	Bardiya	13000	35000	13000	15000	7000	8000	2000	6000
30	Patharaiyaa IS	Kailali	2000	2000	2000	700	1000	400	50	0
31	Mohana IS	Kailali	2000	1000	1200	400	600	200	50	0
32	Mahakali IS	Kanchanpur	11600	28000	11000	4500	9000	4500	200	4000
		Total	325919		295660		199930		53945	40300

1.4 Irrigation Development Policies 2070

पृष्ठभूमि

नेपालमा स्थानीय प्रविधिमा आधारित सिन्चाईको इतिहास निकै पुरानो छ । परम्परागत किसान व्यवस्थित सिचाँइ प्रणालीहरु विश्वस्तरमा नै उदाहरणीय मानिन्छ। परापुर्वक कालदेखिनै नेपाली किसानहरुले स्थानीय प्रविधिको उपयोग गदै खोला नदीमा बाधँ निमार्ण गरी कुलोको माध्यमद्धारा खेत बारीमा पानी पुऱ्याइ सिचाँइ गदै आएका छन । सरकारीस्तरमा वि.स. १९८५ मा चन्द्र नहरको निर्माण गरी देशमा नै पहिलो आधुनिक सिँचाइ प्रणालीको शुरुवात गरिएको थियो । वि.स. २०१३ साल पछि याजेनावद्ध विकासको थालनी पश्चात् नेपाल सरकारले यस क्षत्रेमा प्रसस्त लगानी गर्नुका साथै समयानकुल नीति, योजना तथा कार्यक्रम लागू गर्दै आएको छ । वि.स. २०४६ पछि सरकारद्वारा विभिन्न समयमा जलस्रेात एने , २०४९, सिचाँइ नीति–२०४९, यसमा पथ्रम संशोधन, २०५३, सिँचाइ नीति–२०६०, सिँचाइ नियमावली, २०५६, यसमा पहिलो सशोधन, २०६० जारी हुदै आएको छ । विगतका यस्ता प्रयासकै फलस्वरुप देशमा उपलब्ध सिचाँइ सम्बन्धी प्रविधि, भौतिक सरंचना, जनशक्ति र सरकारी तथा गैर सरकारी सस्थागत क्षमतामा उल्लखेनीय अभिवृद्धि भएको छ। यसरी सिचाँइ क्षेत्रको विकास तथा व्यवस्थापनको कार्यलाइ अग्रगति पद्रान गर्न परिमाजिर्त तथा गतिशील सिचाँइ नीति र्तजुमा गरी कार्यान्वयनमा ल्याउनु आजको आवश्यकता हो । तसर्थ राष्ट्रिय जलस्रोत रणनीति तथा राष्ट्रिय जलयोजनाले सिचाँइ क्षेत्रमा लक्षित गरको उद्देश्यले समेत समेटेर सिचाँइ नीतिन्र २०७० जारी गरिएको छ।

दीर्घकालीन दृष्टिकोण

मुलुकका सम्पुर्ण कृषियोग्य भूमिमा उत्पादकत्वमा योगदान गरी वर्षैभरि दिगो एवं भरपर्दो सिंचाई सुविधा पुऱ्याउने यस नितिको दृष्टिकोण (Vision) रहेको छ।

अवधारणा

यो सिँचाइ नीति देहाय बमोजिमको अवधारणामा आधारित छ :

- वर्षभरिनै सिचाँई सेवा उपलब्ध गराउनका लागि बहेउद्धेश्यीय लगायत सतह सिँचाइ, भूमिगत जल सिँचाइ, नयाँ प्रविधिमा आधारित सिँचाइ, लिफ्ट सिँचाइ प्रणालीको विस्तार, प्रवर्द्धन तथा विकास गर्ने ।
  - विगतमा विकास गरिएका सिचाँइ संरचनाहरुबाट वर्षभरि सिँचाइ सुविधा उपलब्ध गराउनका लागि सभ्यताकाआधारमा अन्तर–जलाधार जल स्थानान्तरण (Inter Basin Water Transfer) हुने आयोजना, जलाशययुक्त (Reservoir Based) आयोजनाको निर्माण गर्नु तथा स्थानीय स्तरमा उपलब्ध सतह तथा भूमिगत जलस्रोतको संयोजनात्मक (Conjunctive Use) उपयोग गर्ने ।
- सिँचाइ प्रणालीको विकास एवं व्यवस्थापनमा जनसहभागितामूलक पद्धतिलाई प्रभावकारी रुपमा लागू गर्न उपभोक्ता संस्थालाई जिम्मेवार र उत्तरदायी तुल्याउने ।
- सिँचाइ क्षेत्रको प्रभावकारीता वृद्धि गर्न संस्थागत सुदृढीकरण तथा जनशक्तिको विकास र परिचालन गर्ने ।
- जनसंख्या वृद्धि, आप्रवासन, जलवायु परिवर्तन तथा जलजन्य प्रकोपका कारण पानीका स्रोत तथा तिनको सिँचाइजन्य उपयोगमा परेको प्रतिकूल प्रभावको अध्ययन गरी अनुकुलन सम्बन्धी कार्यक्रमहरु सँचालन गर्ने ।
- विकेन्द्रीकरणको अवधारणा अनुरुप साना सिँचाइको विकास र व्यवस्थापनमा स्थानीय निकायको क्षमता र संलग्नता वृद्धि गर्ने । (www.doi.gov.np)

1.5 Stages in Project Development

Projects can be initiated from the "top down" or the "bottom up". "Top down" projects are normally formulated from a combination of National or Regional plans, together with an inventory of resources (principally land, water and people). "Bottom up" projects are usually stimulated by local requests. Current emphasis is on "bottom up" projects. However, these naturally need to be considered in the light of the competing uses of resources and the potential benefits. The principal stages shown in Figure 1.1 are described below. For (small) projects, stages may be combined, since extensive and protracted studies will be unjustified.

Stage 1: Project identification

Once the idea has been formulated, the project needs to be identified by name and location, and the general feasibility established. The principal sources of information at this stage will be existing data (mapping, aerial photography, hydrological records, etc.). It is unlikely that new data will be generated through survey at this stage, unless information is particularly short. The types of data often available for this activity includes:

- 1:25000 and/or 1:50000 scales Topographic maps (department of survey)
- General soils, land use and land capability maps (LRMP)
- Existing aerial photography (Google earth)
- Climatological records from nearby meteorological stations (available from DHM)
- Hydrological records from nearby gauging stations
- Social/ demographic records.

Identification will normally involve a site visit which will include:

- An inspection of the general topography and geology
- An inspection of existing agricultural conditions (crops grown, land use, number of farmers, local markets, existing infrastructure, soils etc.)
- In inspection of the potential water sources
- Discussions with local farmers
- Discussions with local government officials

During the field visit, a crude assessment of the project potential and of any particular technical or social problems should be made. It may also be possible to make a rough estimate of the likely development cost in NRs/ha.

By the end of the identification stage, it should be clear, from the technical viewpoint, whether the projects is worth pursuing further or not.

Stage 2: Pre - feasibility study

Once a project has been identified as having potential, a pre - feasibility study should be carried out. This stage may be missed out for small and medium sized projects, and feasibility study would follow directly from a satisfactory identification report.

The sources of information used for identification need to be supplemented by a limited amount of survey work and measurement.

The general features of the study would be as follows:

i. The general topographic mapping should be confirmed by limited spot leveling and distance measurement between key features, to confirm existing mapping and support existing aerial photography.

ii. Preliminary hydrological analyses for the identified sources should be conducted using the available data to estimate the likely magnitude of floods, low season flows, sediment run off etc.

iii. An assessment of land capability and suitability for irrigated agriculture should be made from site inspections and a limited survey.

iv. Possible alternatives for sources works (including groundwater and conjunctive use), conveyance and distribution systems should be developed and examined.

v. Preliminary cropping patterns and associated water requirements should be derived and compared with the rainfall and water sources analyses, to assess the overall water balance and confirm the potential irrigable area.

vi. Possible sources of materials should be identified.

vii. Possible environmental effects of development should be listed and reviewed.

viii. Discussions with local farmers to enlist their willingness to participate and establish land status should be held.

ix. Assessments of roads and access should be made.

x. Assessment of marketing facilities for envisaged crops should be made.

xi. An assessment of socio - economic impact should be made.

xii. A program for further development should be prepared.

xiii. Outline costs and benefits should be estimated.

xiv. The need for further studies should be established.

The pre - feasibility study will provide a guide to further development, and will enable conclusions to be reached about:

- The location and size of the irrigation area
- The agricultural plan
- The irrigation sources to meet crop water needs
- The interest of the participants
- The type of engineering works required and possible alternatives;
- The economics of development

At the end of the pre - feasibility study, it should be possible to decide whether to incorporate the project into the overall financing programs and proceed with a feasibility study.

Stage 3: Feasibility study

The main objective of a feasibility study is to fully assess the feasibility of project implementation from technical, institutional and economic points of view. A feasibility study will normally from the basis for financing by externals funding agencies for by Government.

The general aims of the feasibility study should be

i. To collect and review previous studies.

ii. To collect and appraise existing data and supplement these, as necessary, (through field work) to ensure that the bases for decision making are sound.

iii. To screen alternatives and develop the optimum solution.

iv. To prove that the technical and economic criteria set for projects of the type under consideration are met.

v. To prepare programs for implementation of the project (where viable).

vi. To establish local support for the project.

For the typical irrigation project, the feasibility study would be expected to:

- Develop an appropriate cropping pattern from consideration of alternatives
- Establish needs for processing and marketing of produce
- Establish the institutional arrangements required for scheme management
- Establish the tariffs and cost recovery mechanisms
- Estimate water requirements for crops and other purposes
- Establish the sources and availability of water in average and dry years
- Determine the area which can be irrigated
- Consider and screen alternative solutions for distribution of water
- Prepare and cost outline designs of works
- Establish other infrastructure needs
- Estimate benefits (direct and indirect)
- Carry out economic and financial analyses
- Review environmental, social and economic impacts of project development
- Prepare a program for implementation
- Prepare a financing plan

Data requirements to support a feasibility study depend on the scale and type of projects, but would typically include for a medium/ large project:

- Aerial photography at 1:10000 scale with orthophoto mapping for large projects, of topographic survey at 1:5000
- Site surveys of major structures (e.g. alternative headworks locations)
- Survey of main canal lines
- Soil survey and land capability mapping at 1:25000 (semi - detailed)
- Survey of existing crops, cropping patterns and agricultural practices
- Hydrological/ meteorological data to support analyses of flood frequency, dry year flows, sediment run off, erosion, crop water requirements, etc.

- Geotechnical data at major structures sites
- Permeability tests along rotes of major canals
- Tests on materials at identified sources of construction materials
- Cost and benefit estimating data (e.g. construction costs, crop yields and prices etc.)
- Survey of existing environmental conditions
- Socio - economic and institutional survey
- Details of land ownership, ward boundaries etc. for initial establishment of tertiary units

At this stages, options should be carefully discussed with the beneficiary population, principally, these would include:

- Alignments of primary and secondary canals
- Locations of tertiary head regulators and tertiary unit boundaries
- Principles of distribution and management of water particularly at the tertiary/ quaternary levels.
- Commitment of the farmers to building the tertiary/ quaternary systems (under technical assistance)

Sample areas would be chosen to define and test the proposed arrangements of distribution channels at tertiary/ quaternary levels, and obtain farmers views. Final design of channels at this level is best left until construction of the major distribution channels is nearing completion.

Stage 4: Detailed design and tender documents

The purpose of detailed design is to dimension and specify the works to a sufficiently accurate standard for tenders for construction to be held. This normally implies an accuracy of ±15 % in the quantities of work required. The project works are normally "packaged" into one or a number of contracts, depending on the scale and type of works involved. Provision may also be made for works to be carried out by direct labor. The product of detailed design is a set of tender documents comprising:

- Instructions for tendering
- Conditions of contract
- Specifications
- Bill of quantities
- Drawings

Additionally, cost estimates, design reports and programs should be prepared. Data for design are usually drawn from those obtained for the feasibility study, but may additionally include:

- Strip survey along minor canal alignments
- Site surveys of minor structures
- Additional permeability investigations along canal routes
- Hydraulic model testing of major structures (especially headworks)

Stage 5: Construction and commissioning

This stage involves the physical construction of the required engineering works, and putting these into working order prior to commencement of operations.

An important precursor to this stage is land acquisition, which should be commenced as soon as the alignments decided during the detailed design stage are confirmed.

Works may be carried out by construction contractors or by direct labor. In the former case, pre - and post - construction surveys need to be carried out for measuring the completed quantities of work. Quality control will include:

- Approving contractors' methods of working
- Supervising and testing earthworks, concrete works, etc.
- Sampling and testing materials
- Sampling and testing completed works
- Checking completed works for compliance with specified dimensions and tolerance

A socio - economic baseline study undertaken at the end of construction will be necessary, if the impact of the scheme is to be assessed and used for monitoring and evaluation purposes.

Stage 6: Operation and maintenance

Once construction is complete, the works can be put into operation. At this stage, an operation and maintenance manual is normally produced. This specifies procedures for operating the system under various conditions, and provides schedules for maintenance of the works. An important aspect of operation is performance evaluation and reporting. This allows feedback on planning, design and construction implications for operation and maintenance to be incorporated into future solutions on similar projects. Evaluation should not only focus on shortcomings, but should also highlight positive aspects of performance.

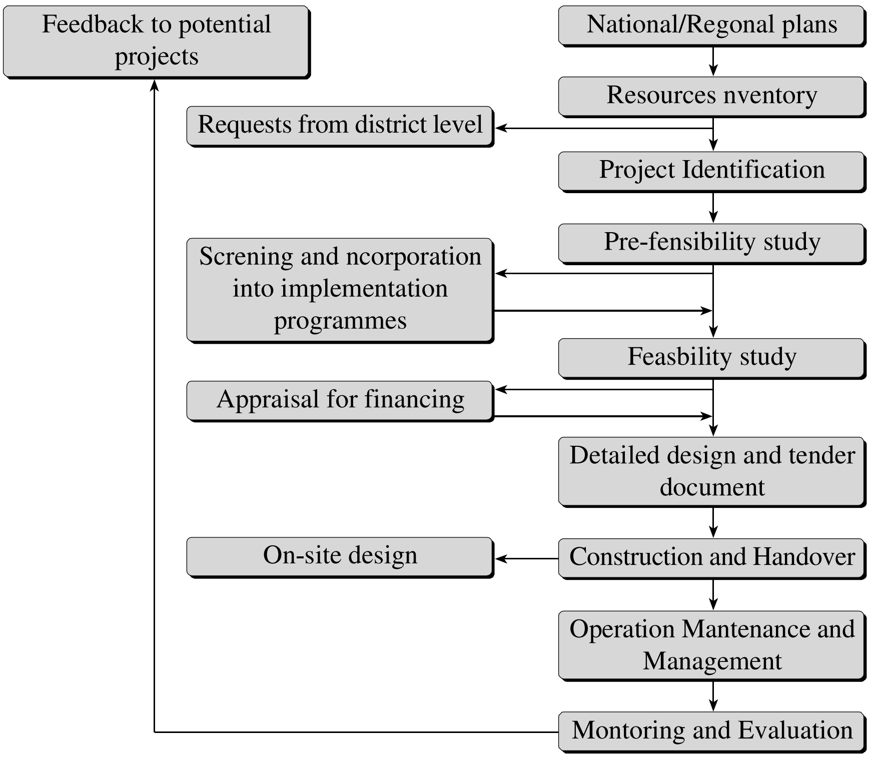

Figure 1.1: Project development stage

1.6 Advantages and Disadvantages of Irrigation

(i) Advantages of irrigation

With the introduction of irrigation, there have been many advantages, as compared to the total dependence on rainfall. These may be enumerated as under:

- **Increase in crop yield:** The production of almost all types of crops can be increased by providing the right amount of later at the right time, depending on its shape of growth. Such a controlled supply of water is possible only through irrigation.
- **Protection from famine:** The availability of irrigation facilities in any region ensures protection against failure of crops or famine due to drought. In regions without irrigation, farmers have to depend only on rains for growing crops and since the rains may not provide enough rainfall required for crop growing every year, the farmers are always faced with a risk.
- **Cultivation of superior crops:** With assured supply of water for irrigation, farmers may think of cultivating superior variety of crops or even other crops which yield high return. Production of these crops in rain-fed areas is not possible because even with the slight unavailability of timely water, these crops would die and all the money invested would be wasted.
- **Elimination of mixed cropping:** In rain-fed areas, farmers have a tendency to cultivate more than one type of crop in the same field such that even if one dies without the required amount of water, at least he would get the yield of the other. However, this reduces the overall production of the field. With assured water by irrigation, the farmer would go for only a single variety of crop in one field at any time, which would increase the yield.
- **Economic development:** With assured irrigation, the farmers get higher returns by way of crop production throughout the year, the government in turn, benefits from the tax collected from the farmers in base of the irrigation facilities extended.
- **Hydro power generation:** Usually, in canal system of irrigation, there are drops or differences in elevation of canal bed level at certain places. Although the drop may not be very high, this difference in elevation can be used successfully to generate electricity for local use.
- **Domestic and industrial water supply:** Some water from the irrigation canals may be utilized for domestic and industrial water supply for nearby areas. Compared to the irrigation water need, the water requirement for domestic and industrial uses is rather small and does not affect the total flow much.

(ii) Disadvantages of irrigation

- **Raising of water table:** Due to the excessive seepage of water through the bed and bank of the canals the water table is in surrounding area may be raised which may constantly saturate the root zone of the crop and soil may develop alkaline property which may be harmful to crops.

- **Formation of marshy land:** Excessive seepage and leakage of water from the irrigation canal may lead to formation of marshy land along the course of canal. These marshy land form the colonies of mosquitoes which may be responsible for diseases.
- **Dampness in weather:** The temperature of Commanded area of an irrigation project may be lowered considerably and the area may become damped. Due to dampness, the people residing around the area may suffer from cold, cough and other such diseases originating from dampness.
- **Loss of valuable land:** Valuable land may get submerged when storage reservoir is formed by constructing barrage or dams and some valuable land also may be lost while constructing irrigation canals.
- **Chances of soil erosion due to flood and over irrigation:** Due to construction of the canal the flood water may enter to the commanded area through the canal which may create soil erosion and inundation in the area.

1.7 Crops, Cropping Periods, Cropping Pattern and Cropping Intensity

In general there are two main crop of Nepal viz. winter and summer Crops. (हिउदेबालि र बर्खेबालि)(*Rabi and Kharif in India*). Normally winter crops are shown in the month of October and are harvested by the end of March, while summer crops are shown in the month of April and are harvested by the end of September. However, the sowing and harvesting times of the crops grown during the two crop seasons may vary from place to place in the country. Moreover, there is usually considerable overlapping of the crops grown in the two seasons.

There are certain crops which have a longer period between their sowing and harvesting times which extends from one crop season to the other. For example sugarcane is sown sometimes during February to March and it is harvested sometimes during November to March of next year. Thus sugarcane takes almost a full year of maturity covering both the crop seasons and hence it is classified as perennial crops. Similarly cotton also requires relatively longer time for maturity. Cotton is sown in May or early June and harvested in December or January next of year. Since cotton requires about eight months for maturity, it is classified as eight month crop.

The above noted classification of the crops is based on the crop season, that is, on the sowing and harvesting times of the crops. However the crops may also be classified on the basis of their irrigation requirements as dry crop and wet crop. Dry crops are those crops which are ordinarily grown without irrigation, but they utilized the moisture stored in the soil during rainfall. The dry crops are grown only in those areas where the irrigation facilities are normally not available, however sometimes dry crops are also irrigated, especially in the years of deficient rainfall, in which case these are known as irrigated dry crop. On the other hand wet crops are those crops which cannot normally be grown without irrigation.

Table: 1.2 Possible cropping pattern (PDSP Manual, 1990)

Region	Alternate	Cropping pattern	Dates		
			Seeded	Transplant/Plant	Harvest
Hills (<1000m amsl)	I	Rice Spring Rice monsoon Wheat	Mar 1 June 2 -	Apr 1 July 2 Nov 2	July 1 Nov 1 Mar 1
	II	Maize Rice Monsoon Wheat	- May 2 -	Mar 2 June 2 Nov 2	June 1 Oct 2 Mar 1
	III	Rice Wheat	June 1 -	July 1 Nov 2	Nov 1 Mar 2
Hills (1000m – 2000m amsl)	I	Rice Wheat	May 1 -	June 1 Nov 2	Oct 1 Apr 1
	II	Vegetable Winter Summer	 - -	 Nov 2 Jul 1	 Mar 1 Oct 2
Terai	I	Rice Spring Rice Monsoon Wheat	Apr 1 July 1 -	May 1 Aug 1 Dec 1	Aug 1 Nov 2 Apr 2
	II	Rice Wheat	June 1 -	July 1 Nov 2	Nov 1 Mar 2
	III	Rice Wheat Mungbean	June 1 - -	July 1 Nov 2 Apr 1	Nov 1 Mar 1 June 1
	IV	Rice Potato	June 1 -	July 1 Nov 2	Nov 1 Apr 1
	V	Rice Maize	May 1 -	June 1 Oct 1	Sep 2 Nov 2
	VI	Jute Rice	- July 1	Mar 1 Aug 1	July 2 Nov 2
	VII	Rice Pulses/ oilseed	Apr 2 -	May 2 Oct 1	Sep 2 Mar 2
	VIII	Sugarcane	-	Feb 1	Dec 2
	IX	Vegetables Winter Summer	 - -	 Nov 2 July1	 Mar 1 Oct 2

Table 1.3: Crop and their required water depth. (FAO, 2017)

Crop	Average water depth (mm)
A: Summer Crops (monsoon Crops)	
– Rice	450 – 700
– Millet	150 – 200
B: Winter Crops (dry Crops)	
– Potato	500 – 700
– Wheat	450 – 650
– Mustard	300 – 400
– Onion	350 – 550
– Cauliflower	350 – 500
– Pea	350 – 500
C: Perennial crops	
– Sugarcane	1500 – 2500
– Banana	1200 – 2200

Cropping Intensity

Cropping intensity is defined the ratio of net area sown by different crops to the total cultivable (cropped) area in a year. For example the cultivated area is 120 ha, now in a particular year if rice has been cropped in 90 ha and wheat is cultivated after rice in 80 ha only, then the annual cropping intensity will be $\frac{90}{120} \times 100 + \frac{80}{120} \times 100 = 141.66\%$.

In Nepal cropping intensity varies from place to place. In Terai (plain area) the cropping intensity may be about 200% (because two crops are grown in whole area in a year). In mountain areas the cropping intensity may be only 100% or less (because only one crop may be grown in a year and not in whole cultivated area). In valley floor of mid - hills like in Panchkhal valley the cropping intensity is 400% (because 4 crops i.e. early paddy, monsoon paddy, potato and potato are grown in a year in whole cultivable area).

1.8 Commanded Areas and Irrigation Intensity

1.8.1 Commanded Area (CA)

Commanded area is defined as the area that can be irrigated by a canal system, the CA may further be classified as under (Reddy, 2010):

(i) Gross Commanded Area (GCA)

This is defined as total area that can be irrigated by a canal system on the perception that unlimited quantity of water is available. It is the total area that may theoretically be served by the irrigation system. But this may include inhibited areas, roads, ponds, uncultivable areas etc. which would not be irrigated.

(ii) Cultivable Commanded Area (CCA)

This is the actually irrigated area within the GCA. However, the entire CCA is never put under cultivation during any crop season due to the following reasons:

- The required quantity of water, fertilizer, etc. may not be available to cultivate the entire CCA at a particular point of time. Thus, this is a physical constraint.
- The land may be kept fallow that is without cultivation for one or more crop seasons to increase the fertility of the soil. This is a cultural decision.
- Due to high water table in some areas of the CCA irrigated water may not be applied as the crops get enough water from the saturation provide to the surface water table.

During any crop season, only a part of the CCA is put under cultivation and this area is termed as cultivable cultivated area. The remaining area which is not cultivated during a crop season is conversely termed as cultivable uncultivated area.

GCA= CCA + uncultivable area

(iii) Net commanded area (NCA)

Net Commanded area is the cultivable commanded area less than the area of canal network, supply ditches, bunds constructed in the field etc. included in the cultivable Commanded area.

NCA = CCA – Canal related structures – non irrigable areas

1.8.2 Intensity of Irrigation

Intensity of irrigation is defined as the percentage of cultivable Commanded area proposed to be irrigated during either a crop season or during a year. For example, if CCA of an irrigation field is 120 hectares, out of which 90 hectares of land is cultivated during summer season and 60 hectare of land is cultivated during winter season, the intensity of irrigation during summer season will be 90/120x100=75% and the intensity of irrigation during winter season will be 60/120x 100= 50%. However, the yearly intensity of irrigation will be equal to the sum of the two, i.e. equal to 75+50 = 120%. Thus yearly intensity of irrigation can be more than 100%. (Punmia, 2009)

1.9 Methods of Field Irrigation and their Suitability

The design, equipment and technique of replenishing the soil water deficit by applying irrigation water is referred to as irrigation method. Five basic methods of the application of irrigation water to the fields to suit different crops, soil type, topography, water availability and its quality, climatic condition, individual holdings and costs.

1.9.1 System of Irrigation

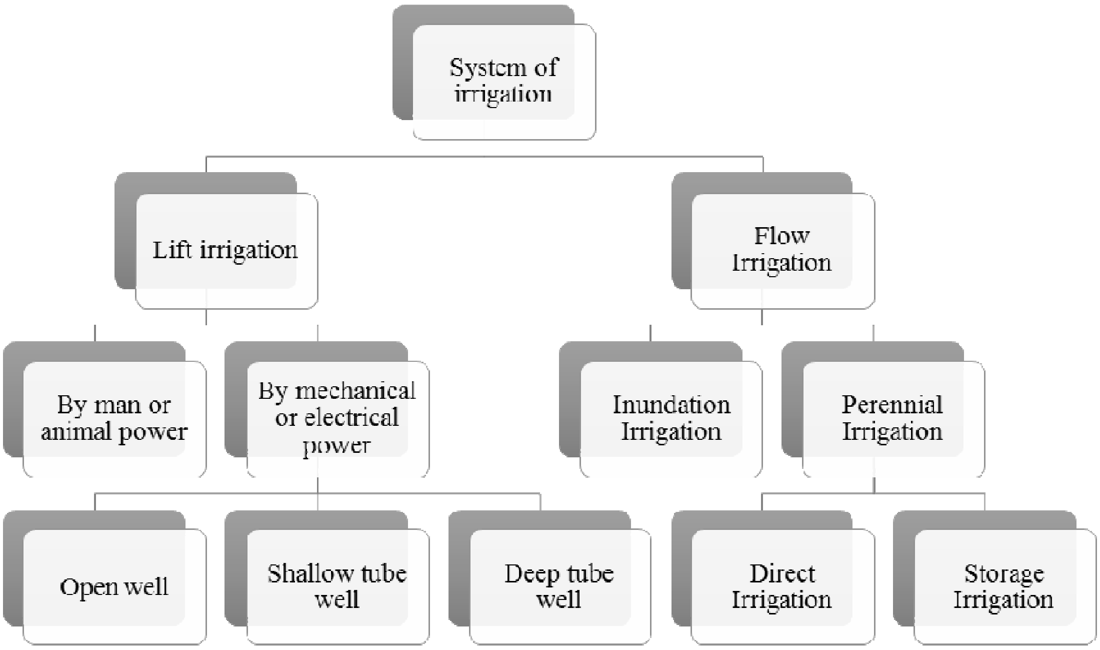

Figure 1.2: System of Irrigation

1. Lift Irrigation

When water is lifted from surface sources or underground sources by man or animal power, mechanical or electrical power and directly supplied to the agricultural land, then it is known as lift irrigation. In this method isolated small area can be irrigated (Basak, 2007). The vast area can't be irrigated in this system. Lift irrigation can be divided in to two groups:

a. Lifting of water by man or animal power

b. Lifting of water by mechanical or electrical power

When mechanical or electrical power are not available in village or the economic condition of farmers is not good enough to afford this expensive method, the lifting of water is done by the following method from the surface sources like pond, lake, river etc.

When mechanical or electrical power are available in village or the can afford the expenditure for the installation of the same, the underground water is lifted by pumps (diesel pump set or electrical pump set) and directly supplied to the agricultural land. The underground water may be available from the following sources:

- Open well
- Shallow tube well
- Deep tube well

Now a days a pumping system of lift irrigation from the shallow or deep tube well is widely practiced. In Nepal Narayani lift irrigation, Marchawar lift irrigation, Waling lift irrigation are example of some lift irrigation projects.

Advantages of Lift Irrigation

The following are the advantages of lift irrigation

i. Farmers can supply water to their field according to the requirement, and hence there is no possibility of over irrigation.
ii. The water table is lowered when the water is lifted from the well thereby reducing chances of water logging in the area.
iii. As water is supplied directly to the fields, there is no water loss due to conveyance.
iv. Initial cost is low as there is no necessity of constructing hydraulic structures.
v. As the loss of water is low, the duty of water is high.
vi. The maintenance cost is low.
vii. More than one crop can be grown in a year on the same land.
viii. Loss of valuable land is prevented as there is no necessity of construction the network of canals.
ix. The water of the well is cooler in hot season and warmer in cold season. This phenomenon is favorable for the crops.

Disadvantages of Lift Irrigation

Following are the disadvantages of lift irrigation

i. In summer the surface water may be dried up and the water table may go down below the suction head. Hence, the lift irrigation from the surface and from the shallow tube well may fail in summer.
ii. If the lifting mechanism (i.e. pump) fails due to mechanical or electrical failure, then water cannot be supplied until the mechanism is restored.
iii. The well water has no silt content. The yield of crop therefore depends on chemical fertilizer, which is costly.
iv. The lift irrigation in consideration with the yield of crop is not cost effective.

2. Flow Irrigation

When water flows under gravitational pull through the artificial canal towards the agricultural land, it is termed as flow irrigation. In this system, the head of the canal should always be at higher elevation than the lad to be irrigated. The following are the different systems of flow irrigation.

(i) Inundation irrigation system

In this system a canal is excavated from the bank of the inundation river(i.e the river which overflows in rainy season but nearly dried up in summer and winter). In this case water flows to the agricultural land in rainy season only. There is no regulator at the head of the canal to control the flow of water. The bed level of the canal is fixed at such level that the water can flow through the canal only when water level of the river raise above the canal bed. Again, the flow of water through the canal stops automatically when the water level of the river falls below the canal bed. So this system of irrigation depends completely on the water level of the river. As there is no regulator at the head of the canal, over irrigation is possible resulting in damaging the crops.

(ii) Perennial system of irrigation

In this system a weir or barrage is constructed across the perennial river (i.e the river which flows throughout the year in its full capacity) to raise the water level on the upstream side or a dam is constructed to form a storage reservoir. Then main canal is constructed on either or both the banks of the river. Regulator is constructed at the head of the canal to control the flow of water is available throughout the year. The perennial system of irrigation may be of the following types;

(a) Direct irrigation system

In this system, a weir is generally constructed across a perennial river to raise the water level on the upstream side up to a certain limit, so that the water can flow through the canal. Here, the water level on the upstream side will remain at a constant height and excess water flows over the weir. Sometimes a barrage is constructed, in place of weir, to regulate the water level on the upstream side. The hydraulic structure which is constructed in direct irrigation system is known as diversion head works.

(b) Storage irrigation system

In this system a dam is constructed across a reverb alley to rom a storage reservoir. The main canals may be taken from both sides of the dam. The flow of water through the canal is controlled by head regulator. This storage reservoir is also known as multipurpose reservoir as it serves the following purposes:

- Irrigation
- Water supply
- Hydro-electric power generation
- Fishery
- Flood control

1.9.2 Methods of Irrigation

The primary objective of any irrigation method is to supply water to soil so that moisture will be readily available at all times for crop growth but without indiscriminately adding to the water table, as well as avoiding influence of soil salinity. An efficient irrigation method is that which best suits local condition such as:-

i. Soil characteristics
ii. Kind of crop and its age
iii. Crop rotation
iv. Topographic condition
v. Available water flow
vi. Sate of soil salinity.
vii. Underground water table condition (high or low)

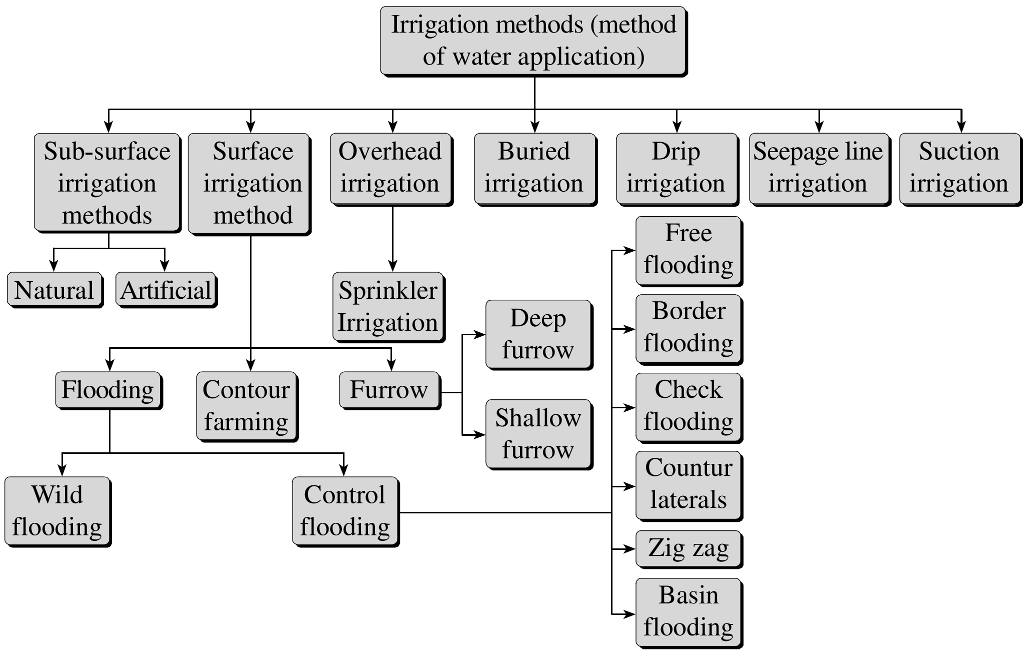

Figure 1.3: Irrigation methods

Requirement of an Irrigation Method

The essential requirements for adoption of any irrigation method are

i. Application is within desirable limits, stream flow is adequate so that the quality of irrigation is such that the depth of wetting and hence the stand of crop are approximately uniform. The irrigator achieves high productivity so that during day's work he can irrigate a large hectares
ii. Afford a uniform water distribution in root zone of a crop with as small as 6 mm applications for light irrigation
iii. Allow heavy uniform application of 15 to 20 cm of water depth and under some conditions as much as 25 cm per irrigation for salt leaching where such a problem exists.
iv. Allow use of large concentrated water flows for reduction of conveyance losses, field channel network and labor cost.
v. Suitable for use with economical conveyance structure
vi. Facilitate mechanized farming
vii. Occupy minimum land under bunds
viii. Inexpensive and economically justifiable.
ix. High efficiency of water application i.e. the ratio of water stored in the root zone to that delivered to the field should be maximum.
x. Minimum wastage of water either through surface runoff or through deep percolation below the root zone of crops.

A suitable irrigation method shall ensure maximum yield at optimum water utilization i.e. conservation of water resources, the choice of an irrigation methods suitable to a specific situation as an important aspect of water management of corps.

1.9.2.1 Subsurface Irrigation

In sub surface or sub-irrigation water is applied beneath the ground by creation and maintaining an artificial water table at some depth, usually 30 – 75 cm, below the ground surface. Moisture moves upward towards the land surface through capillary action to meet requirements of the crops in plant roots. Water is applied through underground distribution system consisting of a properly designed main field ditches, laterals, laid 5 – 30m apart. Water may be obtained from wells, streams, lakes etc. water is introduced into soil profile through open ditches, mole drains or tile drains. Open ditches are preferred because they are relatively inexpensive and suitable for all types of soils. Tiles and mole drains are suitable only for organic soils. Subsurface irrigation requires little field preparation and labor. It entails minimum evaporation loss and surface waste. The irrigation water is essentially required to be of good quality to prevent excessive soil salinity. The flow rate on supply ditches is required to be low to prevent water logging of the field. The use of sub-irrigation is limited because it requires certain soil condition that is the soil is permeable in root zone, underlain by an impervious horizon or high water table.

Essential Requirements

The essential requirement requirements for a successful sub- surface irrigation are

- Availability of adequate supply of good quality water throughout growth period of the crop.
- Fields must be nearly level and smooth. Ground slope is moderate and land is approximately parallel to water table
- Available of layer of permeable soil such as sandy loam or loam immediately below the surface soil to permit free and rapid movement of water laterally and vertically

- Availability of a relatively impervious layer at 2 to 3 m in the substratum to prevent deep percolation of water or a permanently high natural water table on which an artificial water table can be built
- A well planned distribution system of main ditches, field laterals etc. which raises the water table to a uniform depth below the ground surface over the entire area.
- Availability of adequate outlet for drainage of the area so irrigated particularly in humid areas
- Subsoil water table is within 2-3 m below the ground surface
- Topographical conditions are uniform
- Soil is capable of lifting moisture from the water table to the root zone. Also the soil permits lateral and downward movement of water. The efficiency of water use depends on soil characteristics, topography and operation and maintenance management. In good system, the efficiency is 70-75 percent.

Classification of Sub-surface Irrigation

The subsurface irrigation may be classified in to

1. Natural sub surface irrigation

This method is applicable to low lying lands where the water table is high and within the capillary reach of root zone of crops. The water table is charged by seepage from irrigation canals. It is better controlled lest it may develop in to water logging conditions. The method, applicable where above conditions exist offers most economical means of raisin crops.

2. Artificial subsurface irrigation

It is a very expensive method. Its use is indicated only under favorable water supply and subsoil conditions for high yielding crops. The water is provide to crops by capillarity through a network of buried perforated pipes which carry water under pressure to percolate into soil.

Advantages and disadvantages of sub-surface irrigation

Advantages	Disadvantages
(i) Minimum water requirement for raising crops (ii) Suitable for most crops (iii) Minimum evaporation and deep percolation losses (iv) High crop yield (v) Most economical method of irrigation (vi) Involves no wastage of land (v) No interference in free movement of farm machinery (vi) Cultivation operation can be carried out without concern for the irrigation period (vii) Little field preparation and labor	(i) Required a special combination of natural conditions so that only limited areas are suitable for irrigation by this method. (ii) There is danger of development of water logging (iii) Possibility of choking of the pipes laid underground (iv) High cost

1.9.2.2 Surface Irrigation

It is the most common type of irrigation. In surf/ace irrigation method, since water is applied to the field in varied quantities and at different times, the flow remains unsteady. The method involves diverting a stream of water from the head of a field into furrows or borders and allowing it to flow downward. Water infiltrates into the soil while traversing the furrow. By subsequent ponding and lateral movement, the soil is restored to its full water holding capacity to a depth that depends on the quality of water introduced, the duration and rate of stream flow, the gradient and the soil structure and texture. Generally under open ditch conveyance and surface irrigation methods, less than one half of the water released reaches plants. Highly efficient irritation can be achieved by an appropriate combination of size of the irrigation stream, uniform application of water, minimum soil erosion, minimum labor requirement, maximum land use, size of and shape of field and use of machinery. The surface irrigation is essentially supplemented with efficient water disposal system.

Advantages	Disadvantages
• Allow use of machinery for land preparation, cultivation and harvesting. • Helps to store the required amount of water in the capillary zone of the soil for supply to the root zone of plants.	• Greater loss of water by surface runoff and deep percolation, larger requirement of water per unit area • Water is lost in infiltration and deep percolation • Low efficiency due to imperfect control over the water flow. • Inferior quality crops with a low yield. • Wasteful use of water compared to better irrigation methods. • Costly and time consuming preparation of land.

Classification of Surface Irrigation

Surface irrigation may be classified as flooding method, furrow method and contour farming method.

1. **Flooding Method**

   In flooding irrigation, water is allowed to cover the surface of land in a continuous sheet, the water standing just long enough in the field for the soil to absorb the water applied to refill the root zone. A properly designed size of irrigation stream aims at proper balance against the intake rate of soil, the total depth of water to be stored in the root zone and the area to be covered so as to give reasonably uniform coverage of water over the entire field the flooding may be

   (i) Wild Flooding

   It is also called uncontrolled flooding. It is the primitive and most inefficient method of irrigation. In this method water is spread over the smooth or flat field without much control over the flow or prior preparation. The water is spread into the field from the ditch excavated either on the contour or up and down the slope. This method is applicable to inundation irrigation system or for pastures or forage crops where water is available in abundance at the highest elevation and is inexpensive or the crop values do not justify adoption of better method.

The water distribution is quite uneven. The method is suitable for all medium of fine texture soils. It has low cost and doesn't interfere with tillage.

The disadvantages of the method are

- Wasteful use of water
- Non-uniform distribution of water
- Excessive soil erosion on steeper slopes
- Require drainage arrangement to reduce ponding

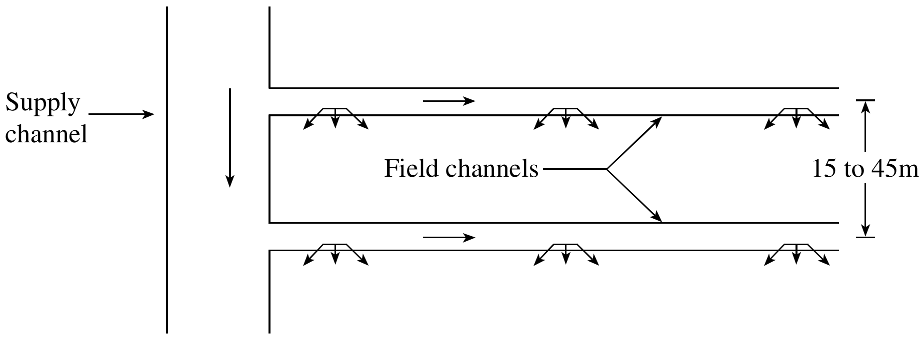

Figure 1.4: Wild or Uncontrolled flooding

(ii) Controlled Flooding

(a) Free Flooding or ordinary flooding

It is the commonly adopted method where irrigation water is in abundance and cheap. The land is divided in to plots of suitable size depending on porosity of soil. Water is spread over the field from watercourse. The irrigation operation begins at the higher area and proceeds towards the lower levels. The flow is stopped when the lower end of the field has received and desired depth of water. The field watercourse is properly spaced, the spacing depends on the topography, soil texture, depth of soil and size of stream. The spreading may vary from less than 15 m to more than 60m. Porous soil requires close spacing than tight soil. The method is most suitable for soils of medium texture and with moderate slopes.

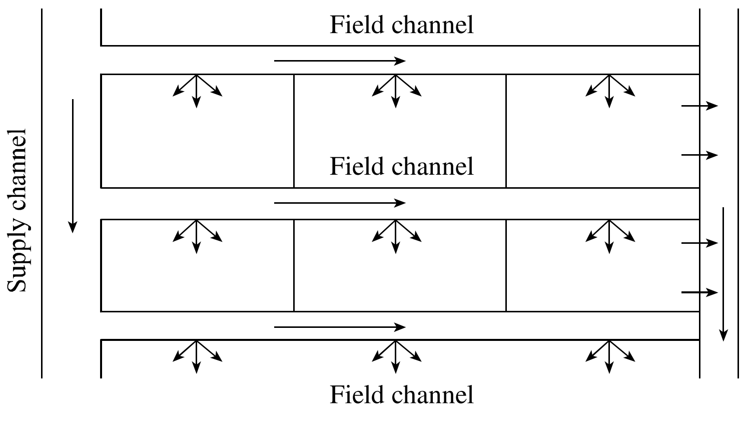

Figure 1.5: Control Flooding

(b) Border Flooding

In this method, the field is divided into narrow strips by low parallel ridges on the sides. The border strip is formed by levelling and grading the land between the ridges. The sizes the strip depends on the types of soil, slope of the land and size of irrigation stream. The width of the strip varies from 5 to 15 m and length varies from for sandy loan soils, 100-120 m for medium loam and 150-300 m for clay loam and clay soils. A longitudinal gradient of 0.02% to 0.05% for clay to clay loam, 0.02-0.4% for medium loam and 0.25% to 0.6% for sandy loam to sandy soil is provided in each strip without any cross slope. The entire land including the rounded border ridges is utilized for cultivation. Thus no land is lost to production. Water is let into each strip in a thin sheet through a set of outlet or siphon pipes underground concrete pipes through riser placed along the upper end of the border. Water flows down the slope along the length of the strip at non –erosive velocity covering the entire width of the strip. Thus sufficient moisture is provided in the soil profile to the entire length of the border. Even distribution of water is ensured if the strip is levelled at a more or less uniform longitudinal slope.

The method is applicable to most soil texture but is best adapted to soils having moderately low to moderately high infiltration rates. It is suitable for high value crops and all close growing crops and is used for some row crops like cotton. In general, it is suitable for hay, pastures and gram crops on lands having slopes up-to 3%.

Advantages	Disadvantages
(i) Utilizes large water streams safely. (ii) Requires less labour and time: low maintenance cost. (iii) Provide uniform wetting and efficient use of water.	(i) Required proper levelling (ii) High initial cost (iv) A large supply of water is needed.

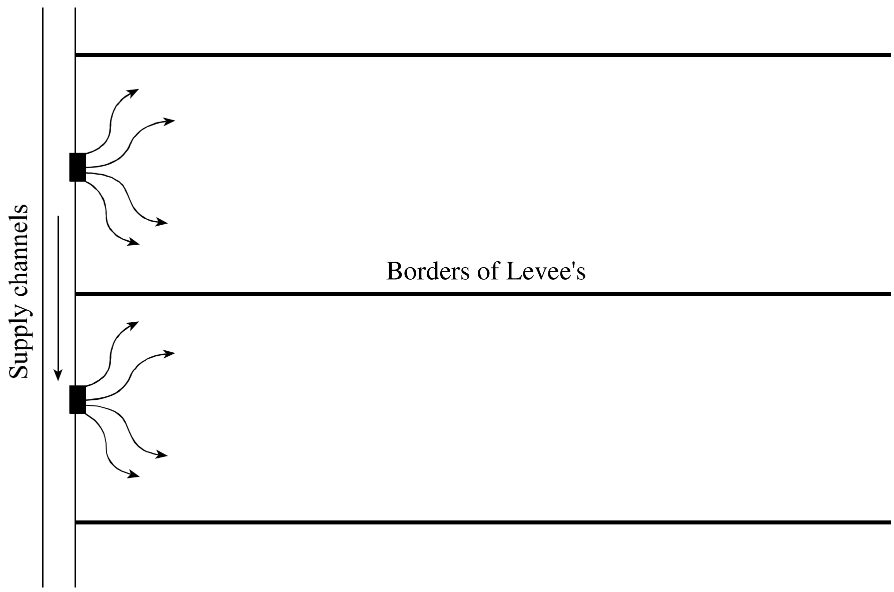

Figure 1.6: Border flooding

(c) Check Flooding

It is widely practiced method of irrigation in Terai. In consist of applying water to relatively level check basins enclosed by small bunds. The checks are square, rectangular or irregular plots. If the ground has some initial slope, levees may follow the contours. The size of check basin may range from 3m × 2m to 30m × 30m or even larger. In orchards, checks are used to irrigate each tree or a group of few trees in a check basin. The levees are 2 – 3 m wide at the base and 25 – 30 cm high in order to avoid obstruction to the movement of farm machinery as also to permit growth of crops over them.

In highly permeable soils, larger size stream is required to quickly impound the check basin with the required quantity of water without allowing deep percolation losses on the side of the irrigation ditch. In highly impermeable soil with low infiltration rate, the water is impounded in the basin and is slowly allowed to infiltrate in the soil. The method is especially suited to fine textured soil which requires holding of water in order to secure the desired penetration. This method is suitable for growing grain and forage crops. The use is generally restricted to relatively smooth lands because of the expenditure involved in levelling the plots.

Advantages	Disadvantages
(i) High irrigation efficiency can be achieved with properly designed check system. (ii) Unskilled labour can be employed as there is no danger of erosion.	(i) High labour requirement (ii) Levees impose restrictions in the use of modern farm machinery (iii) Use is generally restricted to relatively smooth lands because of the expenditure involved in levelling the plots.

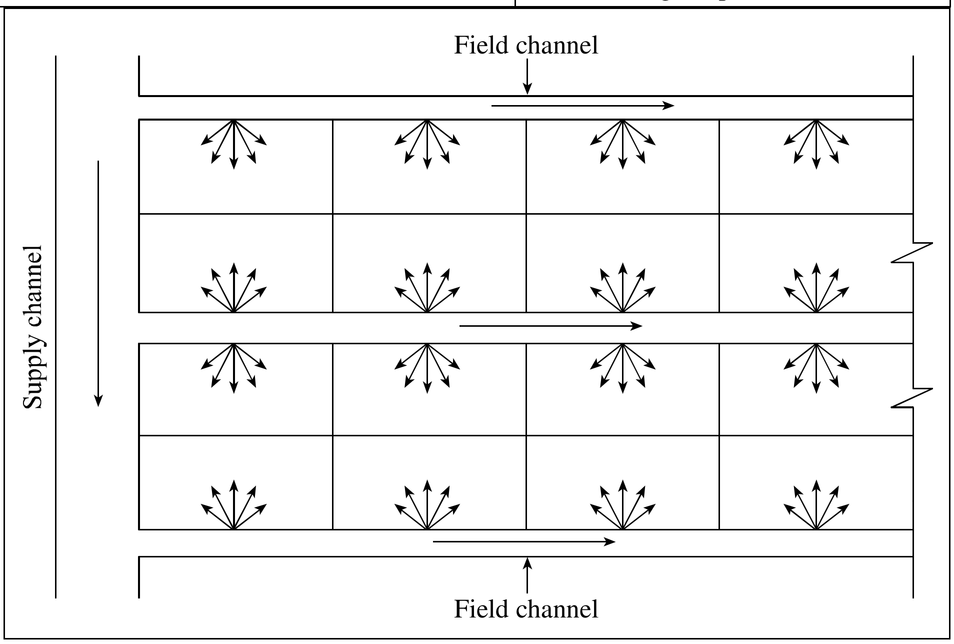

Figure 1.7: Controlled Check Flooding

(d) Contour Lateral Flooding

This method is best suited to steeper terrain. The field is cut by relatively dense network of contour laterals; their spacing depends on the grade of land between two adjacent ditches or laterals, uniformity of slope and type of soil. It may carry from 15 to 50m. The contour ditches may have practically little grade if they are not too long. The water flows through the openings in the laterals. On the other site of contour lateral, a small bund is provided to divert the water from the main to the lateral. The stream size per unit area is the same as in border irrigation.

Contour lateral flooding is adopted on close growing crops in sloping or rolling lands not subject to any degree of levelling necessary for other methods of irrigation. Water flows down slope between closely spaced ditches running along the contour of the land.

Disadvantages

- High labour cost of irrigation
- Not suitable if other method of irrigation can be adopted.

(e) Zig- Zag Flooding

It is a special method of flooding suitable for relatively level fields. But the method is unsuitable for mechanical farming operation. In this method land is divided into square or rectangular plots; each plot is further subdivided with the help of low bunds or levees. Water enters at the upper end of the plot and follows circuitous route to reach the lower end of the plot when the supply is cut off.

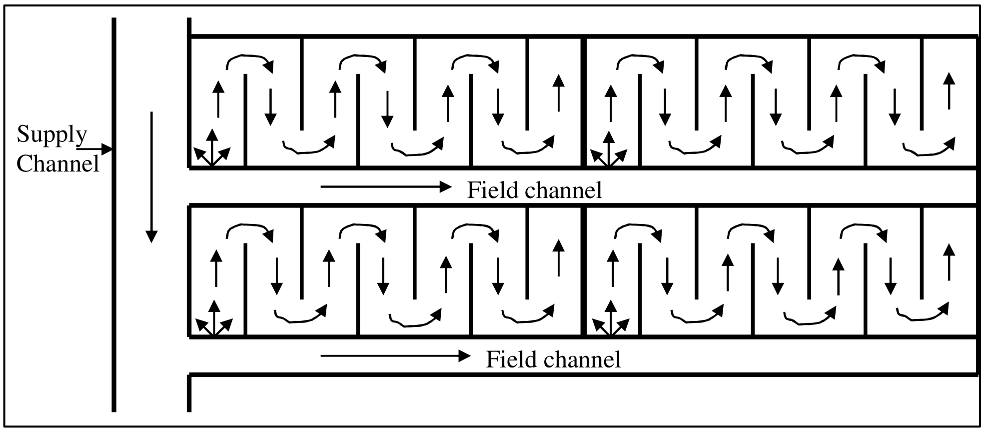

Figure 1.8: Controlled Zig–Zag Flooding

(f) Basin Flooding

It is essentially a check method of flooding adapted for irrigation of orchards. In orchards basins are made around one or more trees depending on the soil condition and topography. For efficient irrigation each basin has to be level, grade less than 0.1% with no cross slope. It is adapted especially to flat lands. Water is filled quickly in the basin and is allowed to percolate into the soil. It is desirable for close growing crops and orchards on medium to coarse textured soils as also rice grown on fine textured, slowly permeable soils. Water is conveyed to the basin or preferably through small ditches feeding individual basins. The merits and demerits of the method are broadly the same as indicated for border irrigation method.

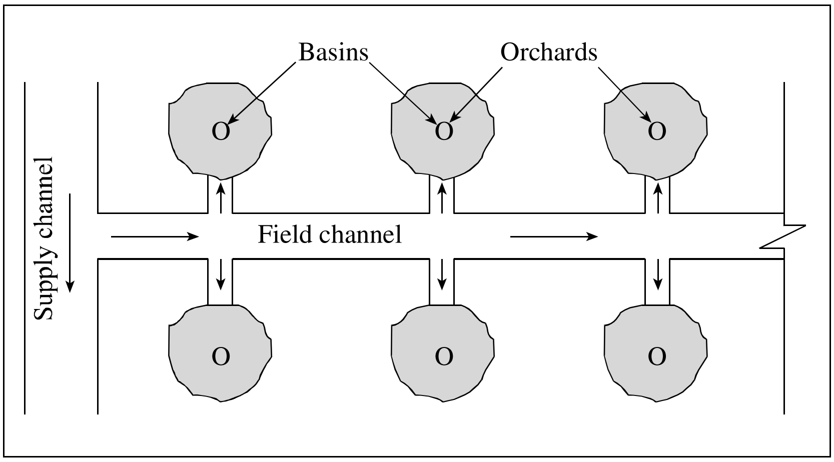

Figure 1.9: Basin Flooding

2. Contour Farming

It is adapted to hilly areas with steep slopes and quickly falling contours. The land is divided into longitudinal curved plots, the bunds of the plots following the contours. The irrigation water stored higher up in some depressions flows between the bunds. The cultivation of crops is along the contour lines instead of the usual down the slope. Contour cultivation reduces runoff and soil loss.

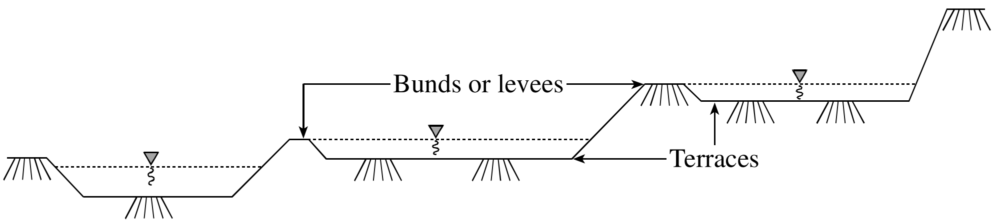

Figure 1.10: Contour Farming

3. Furrow Irrigation

It is the common method of irrigating row crops. A furrow consists of a narrow ditch between the rows of crops. Water is applied in small streams between rows of crops, grown on ridges or in furrows. The characteristic feature of furrow irrigation is that, unlike flooding method, only half to one- fifth of the irrigated land surface is wetted, thereby cutting down evaporation losses. Water flowing in the furrows seeps into the soil and spreads laterally to irrigate areas between the furrows. For distribution of water to furrows, the main ditch is placed in the higher side of the field. The size and shape of the furrows depend on the soul, crop spacing, equipment used for the formation of furrow and intercultivation. Long furrows are advantageous in intercultivation. However, too long furrow results in deep percolation losses and soil erosion near the upper end of the field, and too little water near the tail.

Advantages	Disadvantages
• Low evaporation losses as only one half of irrigated land surface is wetted. • It is possible to cultivate earlier, in heavy soils. • Furrows serve as drainage ways for surface runoff in areas of heavy rainfall. • Especially suitable for crops like maize that are injured by contact with water. • No wastage of land in field ditches. • Fairly efficient in sue of water. • Relatively cheap to construct and maintain. • Efficient in its use of water.	• Not recommended for very light soil having high infiltration capacity. • Ditch may interfere with tillage. • Usually expensive from the consideration of time and labour cost. • Serious erosion hazard • Adequate drainage need to be provided •

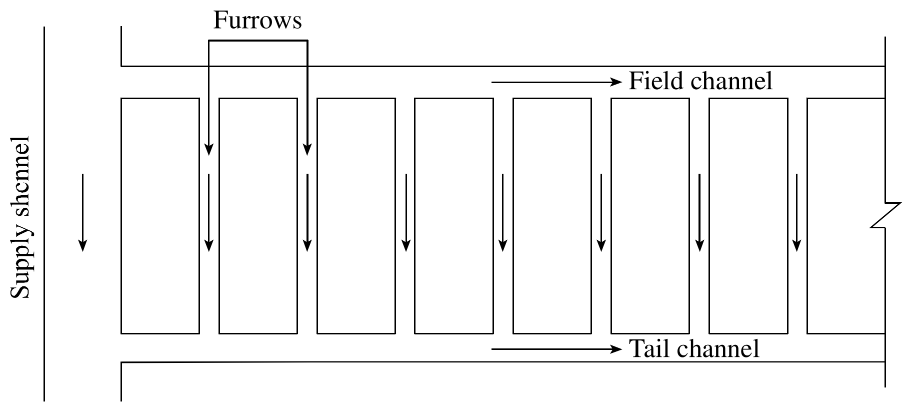

Figure 1.11: Controlled flooding by furrow method

1.9.2.3 Sprinkler Irrigation

Sprinkler irrigation is an improvement over conventional surface irrigation. It simulate natural rainfall to spread water in the form of rain uniformly over the land surface just when needed and as much needed at a uniform pattern and at a rate less than the infiltration rate of the soil so as to avoid surface runoff from irrigation.

Sprinkler is a versatile means of applying water to any crop (except field crops such paddy), soil and topographic condition. It is particularly useful in areas where there is a shortage of water and the hill slopes are steep and can erode with other methods of water application. Surface ditches and prior land preparation are not necessary for sprinkler irrigation. Also the flexible pipes and fittings can be transported to remote areas easily. Sprinkling is suitable for sandy soils or any other soils and topographical conditions where surface irrigation is inefficient or expansive or where erosion of soil may be particularly hazardous. Low rate and amounts of water may be applied such as are required for seed germination, frost protection, delay of fruit budding and cooling of crops in hot weather, fertilizer and soil amending chemicals may be dissolved in the water and applied through the irrigation system.

Advantages	Disadvantages
• Sprinkler provides protection to small plants from wind damage and soil from blowing, when crops such as onions carrots, lettuce and other small seed crop are grown on dry organic soils the soil dries out quickly and seed may be blown away or covered too deeply for germination. When such plants are small they are also easily damaged by windblown soil particles. • Sprinklers provides protection against freezing injury to crops sprinkling is most successful against radiation frost. • Sprinkler can control the soil temperature during period of high incoming radiation soil temperature can rise upto 20^o C above the ambient air temperature.	• Pressure head and sediment free water is required so not suitable at plain land. • Initial cost is relatively high. And regular operation is required due to frequently clogging of system. • Doesn't suitable for paddy crop. • High air flowing area is affected during the operation. • Due to circular spraying of water there is chances of over irrigation and no irrigation at some area.

1.9.2.4 Drip irrigation system

Drip irrigation is a method of applying water directly to plants through a number of low flow rate outlet called emitter or drippers, generally placed at short interval along small tubing. One of the main characteristics of this method is point irrigation as compared to area irrigation with sprinkler or flood irrigation. A network of laterals with drippers supply the water and fertilizer to the plant roots. Water is generally discharged at very low rate in order of 1 – 2 liter per hours, although higher rate up to 8 liter per hours or more can be achieved using special dripper. This type of irrigation can be adopted in the arid regions for fruit and nut trees, grapes, and other vine crops, sugarcane, pineapples, strawberries, flowers and vegetables.

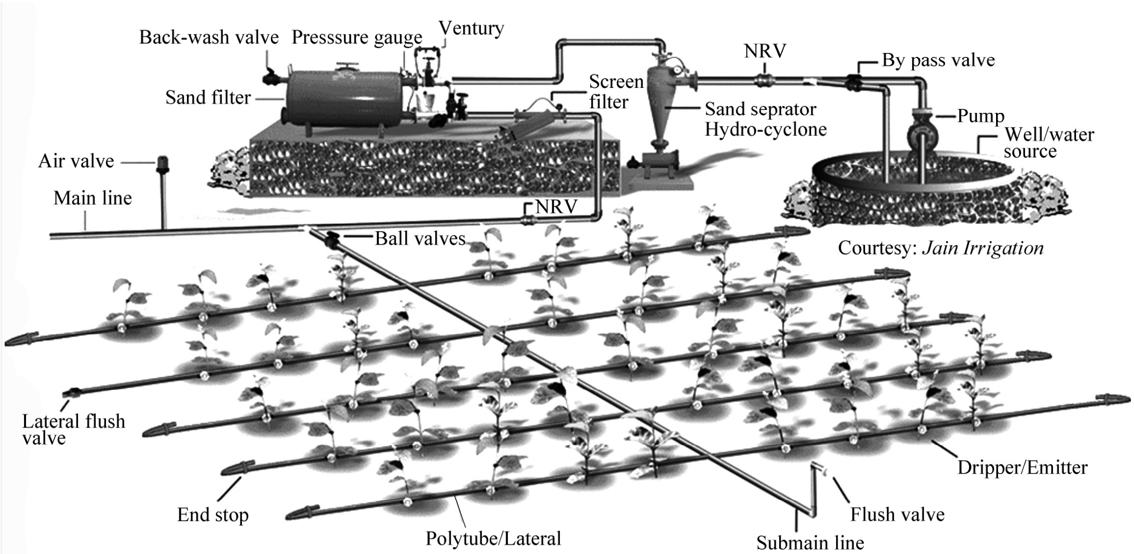

Figure 1.12: Drip irrigation system

Advantages	Disadvantages
• Only the root zone of the plant is supplied with water, with proper system management deep percolation losses can be minimized. • Evaporation losses are minimized because only a portion of the ground area is wetted. • Effective weed control especially for ground area that is not wetted. • Reduced percolation and evaporation losses resulting in greater economy of water use. • Bacteria, fungi and other pests and diseases that depend on the moist environment are reduced as above ground plant parts are completely dry. • Fertilizer and nutrient loss is minimized due to localized application and reduced leaching. • Water application efficiency is high if managed correctly • Field levelling is not necessary. • Fields with irregular shapes are easily accommodated. • Moisture within the root zone can be maintained at field capacity. • Soil type plays less important role in frequency of irrigation. • Soil erosion is lessened. • Weed growth is lessened. • Water distribution is highly uniform, controlled by output of each nozzle. • Labor cost is less than other irrigation methods. • Variation in supply can be regulated by regulating the valves and drippers. • Foliage remains dry, reducing the risk of disease. • Usually operated at lower pressure than other types of pressurized irrigation, reducing energy costs.	• Frequent clogging of dripper; constant supervision may be needed in hill area where the spring water may carry sediment, biological or chemical matter. • This type of irrigation may not be suitable to some crops as salinity can develop as salts tend to accumulate along the fringes of the wetted surface strip. • Root development may be restricted to the wetted soil volume near each emitter. • Not suitable for very closely seeded crops such as carrots, radishes etc. • Initial cost can be more than overhead systems. • The sun can affect the tubes used for drip irrigation, shortening their usable life. • For subsurface drip the irrigator cannot see the water that is applied. This may lead to the farmer either applying too much water (low efficiency) or an insufficient amount of water, this is particularly common for those with less experience with drip irrigation. • Waste of water, time and harvest, if not installed properly. These systems require careful study of all the relevant factors like land topography, soil, water, crop and agro-climatic conditions, and suitability of drip irrigation system and its components. • In lighter soils subsurface drip may be unable to wet the soil surface for germination. Requires careful consideration of the installation depth. • The PVC pipes often suffer from rodent damage, requiring replacement of the entire tube and increasing expenses. • Drip irrigation systems cannot be used for damage control by night frosts (like in the case of sprinkler irrigation systems)

Sustainability of sprinkler and drip irrigation systems in the Hills of Nepal (Hill Irrigation Engineering, 1995)

- Sprinklers and drips need pressure head and sediment free water which is easily available in the Hills of Nepal. In the hilly region the source of water is spring or small stream with small catchment area which have relatively clean water.
- The cropping pattern in the hills are suitable for sprinkler and drip irrigation. Sprinkler can be used for scattered crops like carrot, radish, wheat, mustard, millet etc. which are cultivated in hills. Drip can be used for row crops like tomato, cabbage, cauliflower, chilly etc. which are grown in hills of Nepal.
- These systems require network of pipe lines hence the construction of the canal and other costly structures like headworks, river training works, drops, cross-drainages etc. as in canal irrigation system, are not required and the construction cost for sprinkler and drip is comparatively very low as well as operation and maintenance cost also reduced.
- Due to steep slope in hills the construction of canal is difficult and costly hence the conveyance of water through the pipe system is most suitable in hilly region. The pipes can be easily transported by yak, donkey and/or human effort.
- The flexible tubing, quick connecting couplers and sprinkler heads and lateral pipes can be easily transported to remote areas hence sprinkler and drip are appropriate.
- In case of blockage which can be very frequent in system suing spring water sources, the flexible tubing quick connecting couplers and sprinkler heads can be easily dismantled and refitted after cleaning. Hence operation and maintenance cost is reduced.
- All movable components (the main pipe from the collection tank to the irrigated area will usually be buried and anchored to the ground to prevent theft) can be removed for safety each day at the end of irrigation. (Hills farmers of Nepal live several hours walking distance up hills of their cultivated land.)
- Sprinkler and Drip can also be used on land with slope and undulations of the hilly terrain.

1.10 Planning of Irrigation Projects

Planning is essential in every sphere of development, including irrigation development. Plans give a sense of direction and put time frames on development activities. They help to control expenditure and provide a basic for the evaluation of progress and achievement.

Economic growth in an agricultural sector can be slow because it is influenced by the growth of several other related sectors. It is therefore necessary to set long term strategies for irrigation development and to constantly monitor the effect of a given strategy on the national economy. These strategies can then be translated into short term investment programs depending on the availability of resources. To be effective the plan must fulfill the following objectives:

- It must provide a long term strategy for the development of the irrigation sector that is consistent with availability of land, water and investment resources, and with the development policies of the government.

- It must provide a basis for the preparation of short term investment programs that are consistent with the long term strategy, based on identification and ranking of investment opportunities and on assessment of urgent needs and of the implementation capabilities of government institution.
- It must provide a sound database and planning methodology to facilitate regular updating, to incorporate new data as they become available, to reflect changing policies and priorities, and to reflect actual progress achieved in the irrigation sector.

The process of planning of an irrigation project can be divided into the following two stages:

(i) Preliminary planning (ii) Detailed planning

Preliminary plans, based on available information, are generally approximate but set the course for detailed planning. Based on preliminary planning, the detailed measurements are taken and the detailed plans are prepared. Obviously, detailed plans are more accurate. Alterations in the detailed plans may be necessary at all stages of the project. The preparation of plans of an irrigation project in an undeveloped region is a complicated task and needs the expertise of specialists in areas of engineering, agriculture, soil science, and geology. The following are the main factors which must be determined accurately during the planning stage of an irrigation project:

- Type of project and general plan of irrigation works,
- Location, extent and type of irrigable lands,
- Irrigation requirements for profitable crop production,
- Available water supplies for the project,
- Irrigable (culturable) areas which can be economically supplied with water,
- Types and locations of necessary engineering works,
- Needs for immediate and future drainage,
- Feasibility of hydroelectric power development,
- Cost of storage, irrigation, power, and drainage features,
- Evaluation of probable power, income, and indirect benefits,
- Method of financing the project construction,
- Desirable type of construction and development,
- Probable annual cost of water to the farmers,
- Cost of land preparations and farm distribution systems, and
- Feasible crops, costs of crop production, and probable crop returns.

Most of these elements of project planning are interrelated to some extent. Hence, the studies of the factors listed above should be carried out concurrently so that necessary adjustments can be made promptly as planning progresses. The preliminary planning of an irrigation project consists of collecting and analyzing all available data for the current study, securing additional data needed for preparing preliminary plans for major project features by limited field surveys, and determining the feasibility of the proposed development by making the preliminary study of major features in sufficient detail. While investigations for the preliminary planning of irrigation projects should be conducted with minimum expenditure, the results of the preliminary study must be sufficiently accurate. For preliminary investigations, hydrological studies can be based on the records of stations in the vicinity of the proposed project site. Suitability of land for cultivation purposes can be examined at representative sample areas. Foundation conditions at major irrigation works can be determined from surface and a few subsurface explorations. For detailed planning, accurate data on all aspects of the proposed irrigation project are required to work out the detailed plans and designs of various engineering works and to determine their economic site locations. Physical data needed for detailed planning are collected by topographic and location surveys, land and soil investigations and geological explorations (surface as well as subsurface) at the sites of major engineering works. Results of such surveys are suitably tabulated or plotted for convenient use in design offices and for planning further field work, if necessary. Hydrological data are usually determined by extensive studies of all available records and collecting additional data, if possible. Photographic records of pre-construction (and also during construction) condition at locations of all engineering works and aerial surveys for dams and reservoir sites must be supplemented by accurate ground surveys. Geological explorations are also needed at the sites of dams, reservoirs, and major structures. Such data are useful in studies of water loss due to leakage and foundation designs. Sources of suitable amounts of building material (such as earth material, concrete aggregates, *etc*.) must be located and explored. In case of insufficient supplies at the site, additional sources must be located. Having collected the required data for detailed planning, general plans for irrigation structures are prepared. Such plans are dependent on topography, locations of irrigable areas, available water sources, storage requirements and construction costs. There can be different types of possible feasible plans for a particular project. Advantages and disadvantages of all such possible alternatives must be looked into before arriving at the final plan for the project. Possibilities of using irrigation structures (dams and canal falls) for the development of hydroelectric power should also be examined in project planning. (Asawa, 2005).

1.11 Basin Transfer Projects

Inter – basin transfer or Trans – basin diversion are the terms used to describe man-made conveyance schemes which move water from one river basin where it is available, to another basin where water is less available or could be utilized better for human development. The purpose of such designed schemes can be to alleviate water shortage in the receiving basin and/or to generate electricity through the created head between two basins. Rarely as in the case of the Glory River which diverted water from the Tigris to Euphrates River in modern Iraq, have interbrain transfer been undertaken for political purposes. Since conveyance of water between natural basins are described as both a subtraction at the source and as an addition at the destination, such projects may be controversial in some places and over time; they may also be seen as controversial due to their scale, costs and environmental or developmental impacts. Some proposed inter – basin diversion projects in Nepal are:

- Sunkoshi Kamala Diversion Multipurpose
- Bheri Babai Diversion Multipurpose
- Sankoshi Marine Diversion Multipurpose
- Kaligandaki Tinau Diversion

Exercise 1

1. Define irrigation and drainage. What are the advantages and disadvantages of irrigation?
2. Describe the different types of irrigation methods with neat sketches.
3. Explain status and need of irrigation development in Nepal.
4. Explain methods of field irrigation and their suitability.
5. Define the importance of irrigation in Nepal.
6. What are the challenges and opportunities of irrigation development in Nepal?
7. Discuss about the suitability and appropriateness of drip and sprinkler irrigation system in Nepal.
8. What are the factors which must be determined during the planning stage of an irrigation project?
9. What do you understand by cropping pattern and cropping intensity?
10. Define different commanded area used in irrigation.

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CHAPTER 2

Crop Water Requirement

2.1 Introduction

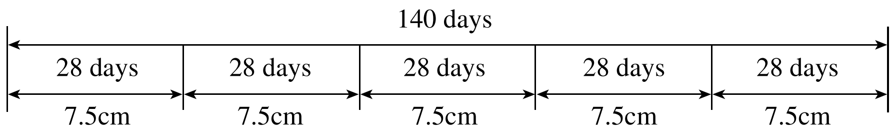
Delta

Each crop requires certain amount of water per hectare for its maturity. If the total amount of water supplied to the crop (from first to last watering) is stored on land without any loss, then there will be a thick layer of water standing on the land. This depth of water layer is known as Delta for the crop. It is denoted by Δ and expressed in cm.

Example 2.1

If wheat requires about 7.5 cm of water after every 28 days, and the base period for wheat is 140 days. Find Δ.

Solution:



Number of watering required = 140/28 = 5

The depth of water required in 140 day = 5 × 7.5 = 37.5 cm

Hence Δ for wheat = 37.5 cm.

Duty

The duty of water is defined as the number of hectares that can be irrigated by constant supply of water at the rate of one cumecs (m^3/sec) throughout the base period. It is expressed in hectares/ cumecs, and is denoted by D. The duty of water is not constant, but it varies with various factors like soil condition, methods of ploughing, methods of application of water etc.

For particular crop,

Area of land = 10 ha

Quantity of water = 5 m^3/sec

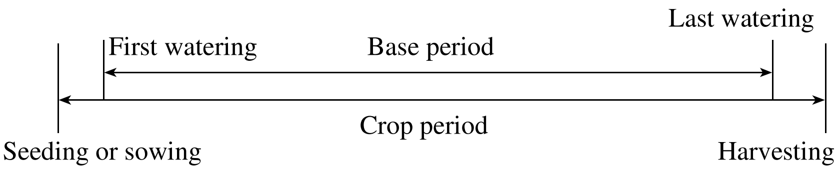
$$\text{Duty} = \frac{10}{5} = 2 \text{ ha/cumecs}$$

Base Period

It is define as the period from the first to the last watering of the crop just before its maturity. It is denoted by B and expressed in no of days.

Crop Period

Crop period is the total time that elapse between the sowing the crop and its harvesting. Thus crop period represent the total time during which crop remains in the field. It is expressed in days.



Relation between Duty and Delta

Let D be the duty of water on the field in ha/cumecs, Δ be the delta or total depth of water in m supplied to a crop growing on the field during the entire base period and B be the base period of the crop in days.

Let 1 cumecs of water be applied to this crop on the field for B days.

Now, the volume of water applied to this crop during B days (V) = $1 \times (60 \times 60 \times 24) \times B = 86400\ m^3$.

By definition of duty (D); one cumecs supplied for B days matures D hectare of land.

This quantity of water V matures D ha. of land or $10^4 \times D$ sq.m. of area.

Total depth of water applied on this land.= $\dfrac{\text{Volume}}{\text{Area}} = \dfrac{86400B}{10^4 D} = \dfrac{8.64B}{D}$ m

This total depth of water is called delta (Δ)= $\dfrac{8.64B}{D}$ m

Example 2.2

Find the Δ for sugarcane when its duty is 730 ha/cumec on the field and base period of crop 110 day.

Solution:

We have the relation, $\Delta = \dfrac{8.64B}{D}$ m

Now, $= \dfrac{8.64 \times 110}{730} = 1.302\text{m} = 130.2$ cm

Example 2.3:

Wheat requires total depth of water 1.230m. The duty at the field is 1400 ha/cumecs, Compute the base period.

Solution:

The base period is given by: $B = \dfrac{\Delta.D}{8.64} = \dfrac{1.23 \times 1400}{8.64} = 199.3$ days.

Example 2.4:

Three distributaries are used for irrigation. The details are given below. Find which one is more efficient.

Discharge	15 m^3/sec	20 m^3/sec	25 m^3/sec
C.C.A	15000 ha	25000 ha	30000 ha
Irrigation intensity	60%	80%	50%
Base period	200 days (cotton crop)	120 days (Wheat crop)	365 days (Sugarcane)

Solution:

1. Area under the cotton crop = 1500 × 0.6 = 9000 ha

   Duty = $\frac{900}{15}$ = 600 ha/cumecs

2. Area under the wheat crop = 25000 × 0.80 = 20000 ha

   Duty = $\frac{20000}{25}$ = 1000 ha/cumecs

3. Area under the sugar cane crop = 30000 × 0.50 = 15000 ha

   Duty = $\frac{15000}{25}$ = 600 ha/cumecs

   The distributary with a higher duty will be more efficient. So distributary II is more efficient.

2.2 Crop Water Requirement (CWR)

Water requirement of crop is the total quantity of water required by crop from the time it is sown to the time it is harvested (crop period). The water requirement of crop will vary with the crop as well as with the place depending upon the variation in climate, types of soil, method of cultivation and useful rainfalls.

Evapotranspiration

Evapotranspiration is actually the combination of two terms – evaporation and transpiration. The first of these, that is, **evaporation** is the process of liquid converting into vapour, through wind action and solar radiation and returning to the atmosphere. Evaporation is the cause of loss of water from open bodies of water, such as lakes, rivers, the oceans and the land surface. It is interesting to note that ocean evaporation provides approximately 90 percent of the earth's precipitation.

Transpiration is the process by which water molecules leaves the body of a living plant and escapes to the atmosphere. The water is drawn up by the plant root system and part of that is lost through the tissues of plant leaf (through the stomata). In areas of abundant rainfall, transpiration is fairly constant with variations occurring primarily in the length of each plants growing season. However, transpiration in dry areas varies greatly with the root depth.

Factor affecting Evapotranspirations (Consmptive use of water)

1. Evaporation which depends upon humidity.
2. Surrounding temperature.
3. Growing season of crop and cropping pattern.

4. Precipitation in the area.
5. Irrigation depth or the depth of water applied for irrigation.
6. Wind velocity.
7. Soil and topography.
8. Irrigation practices and methods of irrigations.

Evapotranspiration, therefore, includes all evaporation from water and land surfaces, as well as transpiration from plants.

Determination of Evapotranspiration for reference crop (Hill Irrigation Engineering, 1995)

(i) Blaney- Crindle Method

This method is suggested for areas where available climatic data cover air temperature data only (Adama, 2012),

$$ET_o = C\,[P\,(0.46T + 8)] \text{ mm/day}$$

Where ET_o = reference crop evapotranspiration in mm/day for the month considered

T = mean daily temperature in °C over the month considered.

P = Mean daily percentage of total annual daytime hours obtained from tables

C = adjustment factor which depends on minimum relative humidity, sunshine and daytime wind estimates.

(ii) Radiation Method

This method is suggested for areas where available climatic data include measured air temperature and sunshine, cloudiness or radiation but not measured wind and humidity.

$$ET_o = c(W.Rs) \text{ mm/day}$$

Where ET_o= reference crop evapotranspiration in mm/day for the month considered

Rs= Solar radiation in equivalent evaporation in mm/day

W= weighting factor which depends on temperature and altitude

C= adjustment factor which depends on mean humidity and daytime wind conditions.

(iii) Penman Method

For the area where measured data on temperature, humidity and sunshine duration or radiation area available, an adaptation of the penman method is suggested; compared to other method presented it is likely to provide the most satisfactory results.

$$ET_o = C\,[w\,R_n + (1 - w) \times f\,(u) \times (e_a - e_d)]$$

Where ET_o = reference crop evapotranspiration in mm/day

W = temperature-related weighting factor

R_n = Next radiation in equivalent evaporation in mm/day

f(u) = wind relation function

$e_a - e_d$ = difference between the saturation vapor pressure at mean air temperature and mean actual vapor pressure of the air, both in bar.

C = adjustment factor to compensate for the effect of day and night weather conditions.

(iv) Pan Evaporation Method

Evaporation pan provide a measurement of the integrated effect of radiation, wind, temperature and humidity on evaporation from a specific open water surface.

$ET_0 = Kp \times Epan$

Where,

Kp = pan coefficient

Epan = Pan evaporation in mm/day and represents the mean daily value of period considered.

2.3 Irrigation Water Requirement (IWR)

Irrigation Water Requirement is defined as the sum of crop water requirement plus all types of water losses.

IWR= CWR + Losses

(i) Consumptive Irrigation Requirement (CIR)

It is the quantity of water actually required by the plant. If a part of the consumptive use is provided by the natural rainfall, the consumptive irrigation requirement is given by,

CIR = Cu – Re

Cu = Consumptive Use

Re = Effective rainfall

(ii) Net Irrigation Requirement (NIR)

It is defined as the amount of irrigation water required to be delivered at the field to meet the evapotranspiration needs a crops as well as the other needs such as leaching, pre-sowing requirement and nursery water requirement (if any). Thus we have

NIR = CIR + LR + PSR + NWR

Where,

LR = Leaching requirement

PSR = pre-sowing requirement

NWR= Nursery water requirement

(iii) Field Irrigation Requirement (FIR)

It is defined as the amount of water required to meet the net irrigation requirements plus the amount of water lost as surface runoff and through deep percolation.

FIR = NIR + water Application loss

$$FIR = \frac{NIR}{\eta_a}$$

η_a = water application efficiency which accounts for losses of irrigation water by surface runoff and through deep percolation.

(iv) Gross Irrigation Requirement (GIR)

It is defined as the amount of water required to meet the field irrigation requirements plus the amount of irrigation water lost in conveyance through the canal system by evaporation and by seepage.

$$GIR = FIR - \text{Conveyance loss} = \frac{FIR}{\eta_c}$$

η_c = Conveyance efficiency

2.4 Effective Rainfall

Part of the rain water percolates below the root zone of the plants and part of the rain water flows away over the soil surface as run-off. This deep percolation water and run-off water cannot be used by the plants. In other words, part of the rainfall is not effective. The remaining part is stored in the root zone and can be used by the plants. This remaining part is the so-called effective rainfall. The factors which influence which part is effective and which part is not effective include the climate, the soil texture, the soil structure and the depth of the root zone.

The actual estimate of the effective rainfall is cumbersome, however approximately it can be estimated as 80 % of the measured rainfall for the designed time.

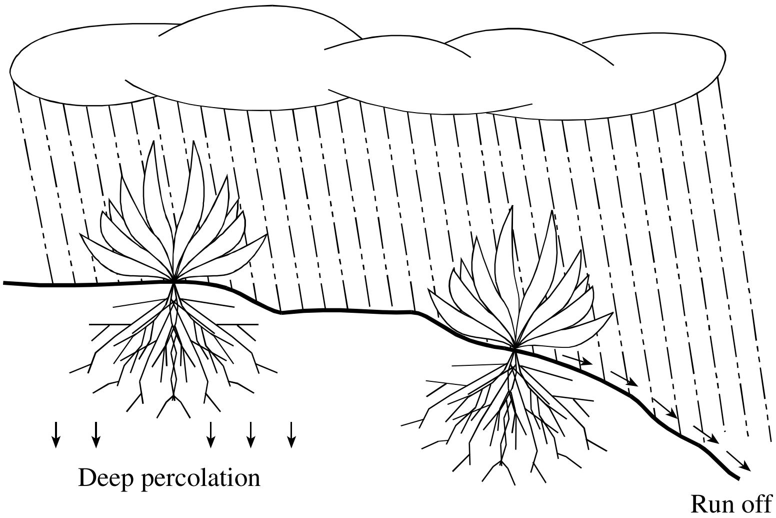


Figure 2.1: Process of rainfall and effective rainfall for crops

Example 2.5

An irrigation project has 6000 Ha of CCA and ET_o is 150 mm/day, effective rainfall is 30 mm/month and overall efficiency of the project is 30%. Calculate the irrigation demands in cumecs for Rice Crop.

Solution:

Here,

CCA= 6000 Ha

ET_o= 150mm/day

Effective Rainfall= 30mm/month=30/30=1 mm/day

Efficiency= 30%

We have, $ET_{crop} = ET_o \times Kc$

For rice $K_c = 1.1 - 1.3$

Take $K_c = 1.1$

$ET_{crop} = 1.1 \times 150 = 165$ mm/day

Irrigation requirement = $ET_{crop} - R_e = 164$ mm/day

Irrigation demand= $\dfrac{164 \times 10^{-3}}{86400} \times 6000 \times 10^4 = 113.89\ m^3/sec$

Project efficiency is 30%

So, Irrigation demand = $\dfrac{113.89}{0.3} = 379.63\ m^3/sec$

2.5 Water Loss due to Seepage and Evaporation

When water comes in contact with an earthen surface, whether artificial or natural, the surface absorbs water. This absorbed water percolates deep into the ground and is the main cause of the loss of water carried by a canal. In addition, some canal water is also lost due to evaporation. The loss due to evaporation is about 10 per cent of the quantity lost due to seepage. The seepage loss varies with the type of the material through which the canal runs. Obviously, the loss is greater in coarse sand and gravel, less in loam, and still less in clay soil. If the canal carries silt-laden water, the pores of the soil are sealed in course of time and the canal seepage reduces with time. In almost all cases, the seepage loss constitutes an important factor which must be accounted for in determining the water requirements of a canal.

Between the headworks of a canal and the watercourses, the loss of water on account of seepage and evaporation is considerable. This loss may be of the order of 20 to 50 per cent of water diverted at the headworks depending upon the type of soil through which canal runs and the climatic conditions of the region. For the purpose of estimating the water requirements of a canal, the total loss due to evaporation and seepage, also known as conveyance loss is expressed as m^3/sec per million square meters of either wetted perimeter or the exposed water surface area.

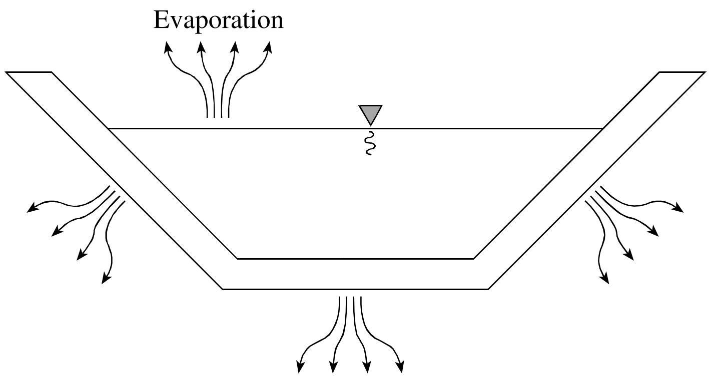


Figure 2.2: Seepage loss and deep percolation

2.6 Operational Water Requirement for Paddy Crops

Paddy cultivation in the hills requires water for land preparation, initial flooding of field, and for percolation losses when the fields are flooded. During land preparation the plough layer must be saturated and the soil puddled. The amount of water needed for this operation depends on the soil types and the initial moisture content of the soil. For spring paddy, 150mm of water is suggested while for main paddy 110mm is considered adequate. If follow paddy is grown after spring paddy the soil is the soil is already in a good state of preparation can be reduced to 55mm. For all paddy crops an initial amount of 100mm of water is required for initial flooding after land preparation. Percolation loss in the paddy fields is usually assumed to be 3 to 5 mm per day. However, these values apply to ideal soil having a heavy clay content. Soil in the hills of Nepal, are somewhat coarse textured and percolation losses of these soils can increase up to 10- 20 mm/day. Considering that each 5 mm of water lost per day is equivalent to about 0.6 l/s/ha it is obvious that percolation losses are an important part in the total field water requirement for paddy. It is therefore recommended that infiltration values (percolation) are actually measure in the field. If no infiltration measurements are available the following average suggested values can be used.

Table 2:1 average percolation for different soil (mm/day)

Soil type	Percolation rate
Clay and clay – loams	3
Loams , silt – loams, or very fine sandy loam	10
Sandy – loams	16

2.7 Soil – Moisture – Irrigation Relationship

Both soil and water are essential for plant growth. The soil provides a structural base to the plants and allows the root system (the foundation of the plant) to spread and get a strong hold. The pores of the soil within the root zone hold moisture which clings to the soil particles by surface tension in the driest state or may fill up the pores partially or fully saturating with it useful nutrients dissolved in water, essential for the growth of the plants. The roots of most plants also require oxygen for respiration. Hence, full saturation of the soil pores leads to restricted root growth for these plants. (There are exceptions, though, like the rice plant, in which the supply of oxygen to the roots is made from the leaves through aerenchyma cells which are continuous from the leaves to the roots).

Since irrigation practice is essentially, an adequate and timely supply of water to the plant root zone for optimum crop yield, the study of the inter relationship between soil pores, its water-holding capacity and plant water absorption rate is fundamentally important. Though a study in detail would mostly be of importance to an agricultural scientist, in this lesson we discuss the essentials which are important to a water resources engineer contemplating the development of a commanded area through scientifically designed irrigation system.

2.7.1 Soil properties

Soil is a complex mass of mineral and organic particles. The important properties that classify soil according to its relevance to making crop production (which in turn affects the decision making process of irrigation engineering) are:

- Soil texture
- Soil structure

Soil Texture

This refers to the relative sizes of soil particles in a given soil. According to their sizes, soil particles are grouped into gravel, sand, silt and day. The relative proportions of sand, silt and clay is a soil mass determines the soil texture. Figure 2.3 presents the textural classification of 12 main classes as identified by the US department of agriculture, which is also followed in Nepal.

According to textural gradations a soil may be broadly classified as:

- Open or light textural soils: these are mainly coarse or sandy with low content of silt and clay.
- Medium textured soils: these contain sand, silt and clay in sizeable proportions, like loamy soil.
- Tight or heavy textured soils: these contain high proportion of clay.

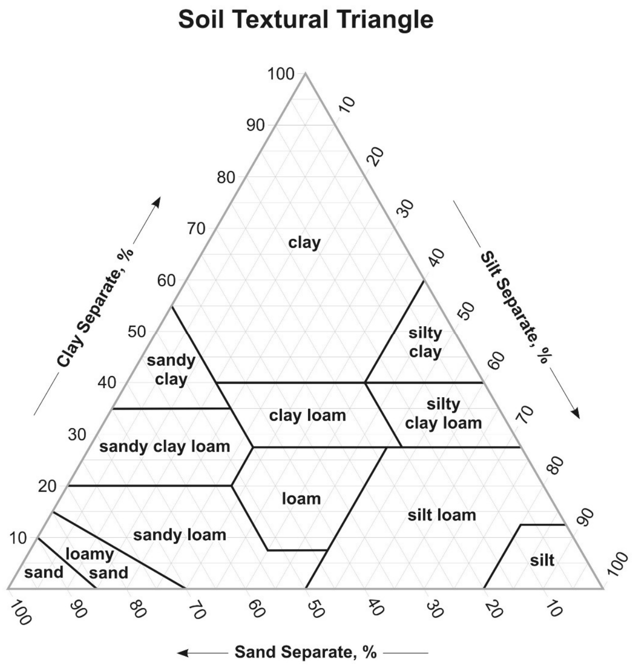


Figure 2.3: USDA Textural Classification of Soil

Soil Structure:

This refers to the arrangement of soil particles and aggregates with respect to each other. Aggregates are groups of individual soil particles adhering together. Soil structure is recognized as one of the most important properties of soil mass, since it influences aeration, permeability, water holding capacity, etc. The classification of soil structure is done according to three indicators as:-

- Type: there are four types of primary structures-platy, prism-like, block like and spheroidal.
- Class: there are five recognized classes in each of the primary types. These are very fine, fine, medium, coarse and very coarse.
- Grade: this represents the degree of aggradation that is the proportion between aggregate and non-aggregated material that results when the aggregates are displaced or gently crushed. Grades are termed as structure less, weak, moderate, strong and very strong depending on the stability of the aggregates when disturbed.

Root Zone Depth

It is define as the maximum depth of soil in which the crop spreads its root system. From this depth the soil absorbs water.

2.7.2 Classification of Soil Water

As stated earlier, water may occur in the soil pores in varying proportions. Some of the definitions related to the water held in the soil pores are as follows:

- **Gravitational water:** A soil sample saturated with water and left to drain the excess out by gravity holds on to a certain amount of water. The volume of water that could easily drain off is termed as the gravitational water. This water is not available for plants use as it drains off rapidly from the root zone.
- **Capillary water:** The water content retained in the soil after the gravitational water has drained off from the soil is known as the capillary water. This water is held in the soil by surface tension. Plant roots gradually absorb the capillary water and thus constitute the principle source of water for plant growth.
- **Hygroscopic water:** The water that an oven dry sample of soil absorbs when exposed to moist air is termed as hygroscopic water. It is held as a very thin film over the surface of the soil particles and is under tremendous negative (gauge) pressure. This water is not available to plants.

The above definitions of the soil water are based on physical factors. Some properties of soil water are not directly related to the above significance to plant growth.

2.7.3 Soil Water Contents

For a particular soil, certain soil water proportions are defined which dictate whether the water is available or not for plant growth. These are called the soil water constants, which are described below.

- **Saturation capacity:** This is the total water content of the soil when all the pores of the soil are filled with water. It is also termed as the maximum water holding capacity of the soil. At saturation capacity, the soil moisture tension is almost equal to zero.
- **Field capacity:** This is the water retained by an initially saturated soil against the force of gravity. Hence, as the gravitational water gets drained off from the soil, it is said to reach the field capacity. At field capacity, the macro-pores of the soil are drained off, but water is retained in the micro-pores. Though the soil moisture tension at field capacity varies from soil to soil, it is normally between 1/10 (for clayey soils) to 1/3 (for sandy soils) atmospheres.

  Field capacity is expressed as the ratio of weight of water contained in the soil to the weight of the dry soil retaining that water i.e.

  $$\text{Field capacity} = \frac{\text{Weight of water retained in a certaion volume of soil}}{\text{Weight of the same volume of dry soil}} \times 100\%$$

  If 1m^2 area and d meter depth of root zone, then the volume of soil is $1 \times d = d\text{m}^3$

  If dry unit weight of soil is γ_d KN/m^3, then

  The weight of $d\text{m}^3$ of soil is $\gamma_d \times d$ KN

  $$\text{Field capacity} = \frac{\text{Weight of water retained in a certaion volume of soil}}{\gamma_d \times d} \times 100\%$$

Weight of water retained in unit area of soil = FC × γ_d × d KN/m^2

Volume of water stored in unit area of soil= $\dfrac{FC \times \gamma_d \times d}{\gamma_w}$ m

The total water storage capacity of soil = $\dfrac{FC \times \gamma_d \times d}{\gamma_w}$ m

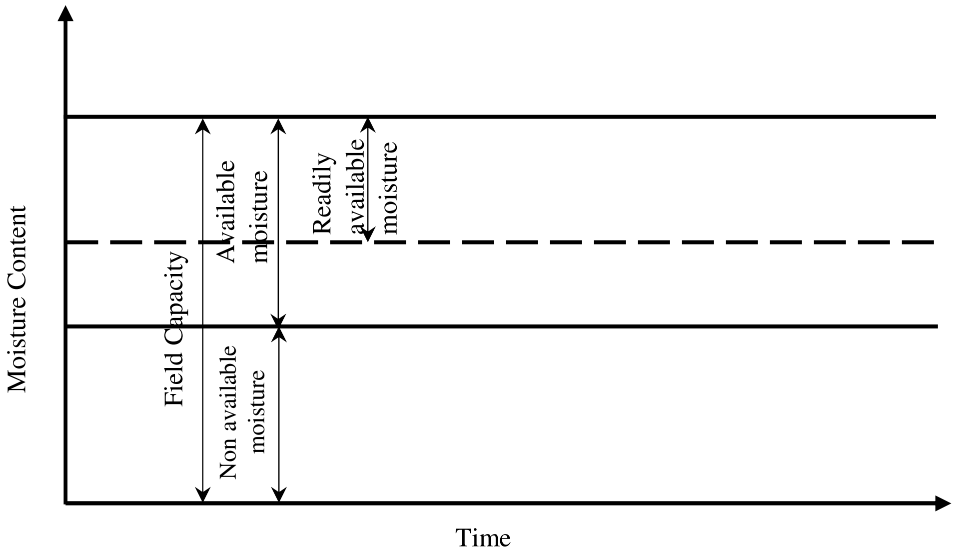
- **Permanent Wilting Point:** Plant roots are able to extract water from a soil matrix, which is saturated up to field capacity. However, as the water extraction proceeds, the moisture content diminishes and the negative (gauge) pressure increases. At one point, the plant cannot extract any further water and thus wilts.

  Two stages of wilting points are recognized and they are:

  – Temporary wilting point: this denotes the soil water content at which the plant wilts at day time, but recovers during right or when water is added to the soil.

  – Ultimate wilting point: at such a soil water content, the plant wilts and fails to regain life even after addition of water to soil.

  It must be noted that the above water contents are expressed as percentage of water held in the soil pores, compared to a fully saturated soil.

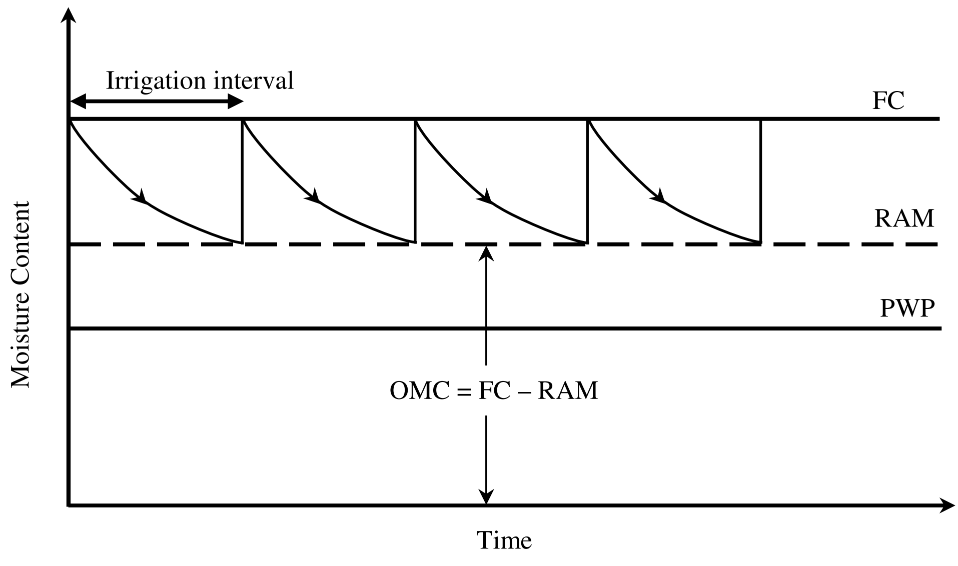


2.8 Depth and Frequency of Irrigation

The irrigation water should be supplied as soon as the moisture falls up to this optimum level (fixing irrigation frequency) and its quantity should be just sufficient to bring the moisture content up to its field capacity, making allowance for application less (thus fixing water depth). The following factors may be taken into consideration for scheduling of irrigation:

- Precipitation rate of the irrigation equipment – how quickly the water is applied, often expressed in inches or mm per hour.
- Distribution uniformity of the irrigation system – how uniformly the water is applied, expressed as a percentage, the higher the number, the more uniform.

- Soil infiltration rate- how quickly the water is absorbed by the soil, the rate of which also decreases as the soil becomes wetter, also often expressed in inches or mm per hour.
- Slope (topography) of the land being irrigated as this affects how quickly runoff occurs, often expressed as a percentage, i.e. distance of fall divided by 100 units of horizontal distance (1 m of fall per 100m would be 1%).
- Soil available water capacity, expressed in units of water per unit of soil, i.e. inches of water per foot of soil.
- Effective rooting depth of the plants to be watered, which affects how much water can be stored in the soil and made available to the plants.
- Current watering requirements of the plant (which may be estimated by calculating evapotranspiration, (or ET), often expressed in mm per day.
- Amount of time in which water or labor may be available for irrigation.
- Amount of allowable moisture stress which may be placed on the plant. For high value vegetable crops, this may mean no allowable stress, while for a lawn some stress would be allowable, since the goal would not be to maximize production, but merely to keep the lawn green and healthy.
- Timing to take advantage of projected rainfall
- Timing to take advantage of favorable utility rates
- Timing to avoid interfering with other activities such as sporting events, holidays, lawn maintenance, or crop harvesting.



The frequency of irrigation should be worked out in advance so that it can be applied in proper intervals. The frequency of irrigation may be ascertained by the following expressions

Depth of water to be applied in each watering = $\frac{\gamma_s \times d}{\gamma_w} \times [FC - OMC] = \frac{\gamma_s}{\gamma_w} \times d \times RAM$

Frequency of Irrigation = $\frac{\text{Allowable soil moisture depletion}}{\text{Rate of consumptive use}}$

Here FC stands for Field Capacity and OMC stands for Optimum moisture Content and d is root zone depth.

Example 2.6

The field capacity of a certain soil is 20% and its apparent specific gravity is 1.6. Before applying irrigation water, a wet sample of soil was taken and its mass was found as 150 gm. The same sample weighed as 136 gm after oven drying. Determine the depth of water that must be applied to irrigate the soil to a depth of 0.9m.

Solution:

Moisture content before irrigation = $\frac{150 - 136}{136} \times 100 = 10.29\%$

$\therefore$ Depth of water required to bring the moisture up to field capacity

$= \frac{\gamma d}{\gamma w} \times d$ [FC - Moisture content before irrigation]

$= 1.6 \times 0.9\ [0.2 - 0.1029]$

$= 0.14\text{m} = 140$ mm.

Example 2.7

During a particular stage of the growth of a crop, consumptive use of water is 2.8 mm/day. Determine the interval in days between irrigations, and depth of water to be applied when the amount of water available in the soil is:

(i) 25%, (ii) 50%, (iii) 75% (iv) 0% of the maximum depth of available water in the root zone which is 80mm. assume irrigation efficiency to be 65%.

Field Capacity

Solution:

(i) Frequency of irrigation = $\frac{80 \times (1 - 0.25)}{2.8} = 21.43$ days = 21 days (say)

(ii) Depth of water to be applied = $\frac{80 \times (1 - 0.25)}{0.65} = 92.31$ mm = 93.00 mm (say)

Other calculations have been shown in the following table:

	Amount of soil moisture depleted to			
	25%	50%	75%	0%
Frequency of irrigation (days)	21	14	7	28
Depth of water to be applied (mm)	93	62	31	124

Example 2.8:

After how many days will supply water to soil in order to ensure sufficient irrigation of the given crop if,

i. Field capacity of the soil= 28%

ii. Permanent wilting point= 13%

iii. Dry density of soil=1.3 gm/cc

iv. Effective depth of root zone=70 cm

v. Daily consumptive use of water for the given crop=12mm

Solution:

Available moisture = Field Capacity – Permanent wilting point

= 28% – 13% = 15%

Assume that Readily Available Moisture = 80% of Available Moisture

RAM = 0.8 × 15% =12%

Optimum moisture = 28% –12% = 16%

It means that the moisture will be filled by irrigation between 16% and 28%.

Depth of water stored in the root zone between these two limits $= \frac{\gamma d}{\gamma w} \times d \times (FC - OMC)$

$$= \frac{1.3}{1} \times 0.7 \times (0.28 - 0.16) = 10.92 \text{ cm}$$

Here, water available for Evapotranspiration =10.92 cm.

Given that; 1.2 cm of water will be utilized by the plant in 1 day.

10.92 cm of water will be utilized by the plant in $= \frac{1 \times 10.92}{1.2} = 9.1$ days ≈ 9 days

Hence, after 9 days, water should be supplied to the given crop.

Example 2.9:

Find the field capacity of a soil for the following data:

(i) Depth of root zone = 2m

(ii) Existing water content = 5%

(iii) Dry density of soil = 1500 kg/m^3

(iv) Water applied to the soil = 600 m^3

(v) Water lost due to Evaporation and Deep Percolation =10%

(vi) Area of land irrigated = 900 m^2

Solution:

Total water applied = 600 m^3

Loss of water = 10%

Water retained in the soil = 600 × 90/100 = 540 m^3

Weight of water retained = 540 × 9810 = 5297400 N

Total dry weight of soil = 900 × 2 × 1500 × 9.81 = 26487000 N

% of water retained in the soil = $\frac{5297400\text{N}}{2648700\text{N}}$ = 0.2 = 20%

Existing water content = 5%

Field capacity = 5% + 20% = 25%

2.9 Irrigation Efficiencies

Irrigation system efficiency, implies minimum loss from the time water is taken from the canal outlet till it becomes available in the root zone for plant utilization. The objective of the efficiency concept is to show where improvements can be made so as to have more efficient irrigation. Method of measurement and evaluation of performance in terms of efficiency are prerequisites for proper use of irrigation water. Generally speaking water use in an irrigation project under development has lower efficiency. However, as water becomes more and more scarce and as the need for it becomes more pressing, its utilization improves. Various concepts of irrigation efficiencies are as under:

1. **Irrigation Efficiency:**

It is defined as the ratio of water output to the water input, i.e., the ratio or percentage of the irrigation water consumed by the crops of an irrigated farm, field or project to water losses occurring at various stages before the plants ultimately utilize it, several partial efficiencies should individually be maximum:

$$\eta_i = \frac{W_c}{W_r} \times 100\%$$

Where,

η_i = irrigation efficiency (%),

W_c = irrigation water consumed by crop during its growth period in an irrigation project, and

W_r = water delivered from canals during the growth period of crops.

In most irrigation projects, the irrigation efficiency ranges between 12 to 34 per cent.

2. **Water Conveyance Efficiency:**

It is a measure of efficiency of water conveyance system from canal network to watercourses and field channels. It is the ratio of water delivered in fields at the outlet head to that diverted into the canal system from the river or reservoir. Water losses occur by water conveyance efficiency as under:

$$\eta_c = \frac{W_f}{W_r} \times 100\%$$

Where, η_c = water conveyance efficiency, percent,

W_f = water delivered to the farm from conveyance system (at field supply channel), and

W_r = water introduced into the conveyance system from the point of diversion.

Water conveyance efficiency is generally low; about 21% losses occur in earthen watercourses only.

3. Water Application Efficiency

It is a measure of efficiency of water application in the field. It is the ratio of volume of water that is stored in the root zone of crops and ultimately consumed by transpiration or evaporation or both to the volume of water actually delivered at the field. Alternatively, it may be defined as the percentage of water applied that can be accounted for as increase in soil moisture in soil as occupied by principal rooting system of the crop. It is also termed as farm efficiency as it takes into account water lost in application at the farm.

$$\eta_a = \frac{W_s}{W_f} \times 100\%$$

Where, η_a = water application efficiency, percent,

W_s = irrigation water stored in the root zone of farm soil, and

W_f= irrigation water delivered to the farm (at field supply channel).

In general, water application efficiency decreases as the amount of water during each irrigation increases. Water losses due to inefficient application of water in the field vary from 28 to 50 percent.

Common sources of losses of irrigation water during application are represented as R_f = surface runoff from the farm, and D_f = deep percolation below the farm root zone soil. Neglecting evaporation losses during application:

$$\eta_f = W_s + D_f + R_f$$

$$\eta_a = \frac{W_f - D_f + R_f}{W_f} \times 100\%$$

4. Water Use Efficiency

Having conveyed water to the point of use and having applied it, the next efficiency concept of concern is the efficiency of water use. It is expressed in kg/ha cm. The proportion of water delivered and beneficially used on the project can be calculated using following formula:

$$\eta_u = \frac{W_u}{W_d} \times 100\%$$

Where, η_u = water use efficiency, percent,

W_u = water beneficially used, and

W_d = water delivered.

Water use efficiency is also defined as (i) crop water use efficiency and (ii) field water efficiency.

(i) Crop Water Use Efficiency: It is the ratio of yield of crop (Y) to the amount of water depleted by crop in evapotranspiration (ET) i.e., Y/ET.

(ii) Field Water Use Efficiency: It is the ratio of yield of crop (Y) to the total amount of water used in the field i.e., Y/WR.

5. Water Storage Efficiency

Also termed as water storage factor. It is defined as the ratio of the water stored in the root depth by irrigation to the water needed in the root depth to bring it to the field capacity

i.e. $\eta_s = \frac{W_s}{W_w} \times 100\%$

Where, η_s = water storage efficiency, percent,

W_s = water stored in the root zone during the irrigation and

W_u = water needed in the root zone prior to irrigation, i.e., field capacity available moisture.

6. Water Distribution Efficiency

It is expressed for distribution efficiency to evaluate the extent to which the water is uniformly distributed i.e. $\eta_d = \left(1 - \frac{d}{D}\right) \times 100\% = 1 - \frac{\text{Average Deviation}}{\text{Average Depth Applied}} \times 100\%$

Where, η_d = water distribution efficiency, percent,

d = average numerical deviation in depth of water stored from average depth stored during irrigation, and

D = average depth of water stored along the run during irrigation.

A water distribution efficiency of 80% means that 10% of water was applied in excess and consequently 10% was deficient in comparison to the average depth of application.

7. Consumptive Use Efficiency:

It is defined as the ratio of consumptive water use by the crop of irrigated farm or project and the irrigation water stored in the root zone of the soil on the farm or the project area. After irrigation water is stored in the soil, it may not be available for use by the crop because water may evaporate from the ground surface or continuously move downward beyond the root zone as it may happen in a wide furrow spacing. The loss of water by deep penetration and by surface evaporation following and irrigation is evaluated from the following expression:

$$\eta_{cu} = \frac{W_{cu}}{W_d} \times 100\%$$

Where, η_{cu} = consumptive use efficiency, percent,

W_{cu} = normal consumptive use of water,

W_d = net amount of water depleted from root zone soil.

Consumptive use efficiency is useful in explaining the difference in crop response from different methods of irrigation.

Example 2.10:

A stream of 125 l/s was diverted from a canal and 100 l/s were delivered to the field. An area of 1.6 ha was irrigated in 8 hrs. Effective depth of root zone was 1.7 m. The runoff loss in the field was 420 m^3. The depth of water penetration varied linearly from 1.7 m at the head end of the field to 1.3 m at the tail end. Available moisture holding capacity of the soil is 20 cm per m depth of soil. Determine water conveyance, water application, water storage and water distribution efficiencies. The irrigation was started at a moisture extraction level of 50% of the available moisture.

Solution:

(i) Water Conveyance efficiency $\eta_c = \frac{W_f}{W_r} \times 100\%$

W_f = 110lps

W_r = 125 lps

η_c = 80%

(ii) Water application efficiency $\eta_a = \frac{W_s}{W_f} \times 100\%$

Water supplied to field during 8 hrs @ 100 lps= 100/1000 × 8 × 60 × 60 = 2880m^3.

Runoff loss in the field= 420 m^3

The water stored in the root zone W_s = 2880 – 420 = 2460 m^3

Now, water application efficiency = $\frac{2460}{2880} \times 100\% = 85.4\%$

(iii) Water storage efficiency $\eta_s = \frac{W_s}{W_w} \times 100\%$

Moisture holding capacity of soil = 20 cm/m × 1.7 m= 34cm

Moisture already available in the root zone

At the time of start of irrigation = 34 × 50%=17cm.

Additional water requirement in the root zone = 34 – 17 =17 cm = 0.17m × 1.6 × 10^4 m^2 = 2720 m^3

But actual water stored in the root zone = 2460 m^3

Water storage efficiency = $\frac{2460}{2720} \times 100\%$

(iv) Water distribution efficiency $\eta_d = \left(1 - \frac{d}{D}\right) \times 100\%$

$$= \left(1 - \frac{\text{Average Deviation}}{\text{Average Depth Applied}}\right) \times 100\%$$

D = Mean depth of water stored in the root zone = $\frac{1.7+1.3}{2}$ =1.5m

$$d = \frac{(1.7 - 1.5) + (1.5 - 1.3)}{2} = 0.2$$

$$\eta_d = \left(1 - \frac{0.2}{1.5}\right) \times 100\% = 86.66\%$$

2.10 Design Discharge of Canal

Kor watering and Kor Depth

The total quantity of water required by a crop is applied through a number of watering at certain interval during the base period of crop. However, the quantity of water required to be applied during each of these watering is not same. In general for all the crop during the first watering after the plant have grown a few centimeter is high, the quantity of water required is more than during the subsequent watering. The first watering after the plant have grown a few centimeter is known as **Kor watering** and the depth of water during this watering is known as **Kor depth**. The kor watering must be done in limited period which is known as **kor period**. Since during kor watering certain quantity of water is required to be applied in a relatively short duration, the discharge capacity of canal supplying irrigation water has to be maximum during this period. For example:-

Total water depth of certain crop= 375 mm

Base period B= 182 day

Duty (D) = $\frac{8.64B}{\Delta}$ m = 4193 ha/cumecs

Area= 1000 Ha

Discharge Q= $\frac{1000}{4193}$ = 0.2384 m^3/sec

Again if;

Kor depth K_d = 135mm

Kor period (K_p) = 3 weeks = 21 days

Duty (D) = $\frac{8.64B}{\Delta}$ m = $\frac{8.64 \text{ x } 21}{0.135}$ m = 1344 Ha/cumecs

Area = 1000 Ha

Q = 0.744 m^3/sec

Then design discharge for canal= 0.744 m^3/sec

Example 2.11:

The gross area of irrigation project is 50000 ha. Out of these about 5000 ha have been utilized for construction of dwellings roads, bridges etc. The area to be cultivate during Rabi (winter season) is 25000 ha and during Kharif (summer) is 24000 ha. If the duty of canal water for winter crop is 5000 ha/cumecs and for summer crops is 3000 ha/cumecs, find the design discharge for the canal after giving 10% allowance for peak discharge and loss of water in transit. What would be canal intensity of irrigation?

Solution:

Here,

GCA = 50,000 ha

CCA = 50000 – 5000 = 45000 ha

Kharif (summer)

Area of cultivation = 24000 Ha

Duty (D)= 3000 ha/cumecs

Discharge (Q) = 24000/3000 = 8 cumecs

Rabi (winter)

Area of cultivation = 25000 ha.

Duty (D) = 5000 ha/cumecs

Discharge (Q) = 5 cumecs

Take higher value for design discharge Qd= 8 + 8 × 10%= 8.8 m^3/sec

CCA=45000 Ha

In summer; Intensity of irrigation = $\frac{24000}{45000} \times 100\% = 53.33\%$

In winter; Intensity of irrigation = $\frac{25000}{45000} \times 100\% = 55.55\%$

Now, Irrigation intensity = 53.33% + 55.55% = 108.88%

Example 2.12:

Water course has a CCA of 15000 Ha. The irrigation intensity for crop A = 45% and for Crop B = 40%, both the crops being rabi crops. Crop A has a kor period of 20 days and Crop B has a 15 days. Calculate the discharge of water course if the kor depths of A & B are 10 cm and 16 cm respectively.

Solution:

Here,

CCA= 15000 ha.

Crop A

K_p = 20 days

K_d =10 days

Irrigation Intensity = 45%

We have $(\Delta) = \frac{8.64B}{D}$ m

Duty A= $\frac{8.64 \times 20}{0.1}$ = 1728 ha/cumec

Discharge Q_A= $\frac{\text{CCA} \times \text{Irrigation Intensity}}{\text{Duty of A}}$

$= \frac{15000 \times 0.45}{1728}$ = 0.39 m^3/sec

Crop B

K_p = 15 day

K_d = 16 cm

Irrigation Intensity= 40%

Duty B = $\frac{8.64 \times 15}{0.16}$ = 810 ha/cumec

Discharge Q_B= $\frac{\text{CCA} \times \text{Irrigation Intensity}}{\text{Duty of B}}$

$= \frac{15000 \times 0.4}{810}$ = 0.39 m^3/sec

Since both the crop has same season, design discharge = $Q_A + Q_B$ = 1.13 m^3/sec

Example 2.13:

The culturable commanded area for a distributary channel is 10,000 hectares. The intensity of irrigation is 30 per cent for wheat and 15 per cent for rice. The kor period for wheat is 4 weeks, and for rice 3 weeks. Kor watering depths for wheat and rice are 135 mm and 190 mm, respectively. Estimate the outlet discharge.

Solution:

Quantity	Wheat	Rice
Area to be irrigated (ha) =	10,000 × 30% = 3000	10,000 × 15% = 1500
Outlet factor or Duty $D = \frac{8.64B}{\Delta}$ ha/cumecs =	$= \frac{8.64\,(4 \times 7)}{0.135} = 1792$	$= \frac{8.64\,(3 \times 7)}{0.19} = 954.95$
Outlet discharge (m^3/sec) =	3000/1792 = 1.7	1500/954.95 = 1.6

Since the water demands for wheat and rice are at different times, these are not cumulative. Therefore, the distributary channel should be designed for the larger of the two discharges i.e. 1.7m^3/sec. The above calculations exclude channel losses and the water requirement of other major crops during their kor period.

Example 2.14:

With the following data, FC = 80%, PWP = 35%, root depth= 60 cm, dry density of soil = 1.58 gm/cc, ETc = 5 mm/ day, application efficiency=80%, Conveyance loss = 55% and Distribution loss=65%. Calculate,

i. Available moisture content

ii. Readily available moisture content

iii. Depth of irrigation at the outlet of the field.

iv. Irrigation interval

v. Irrigation requirement at the headwork.

Solution:

i. Available moisture content (AMC) = FC – PWP = 45%

ii. Readily available moisture content (RAM)

Assume RAM = 80% of AMC

Then, RAM = 0.8 × 45%= 36%

iii. Depth of water available for consumptive use $= \frac{\gamma d}{\gamma w} \times d \times RAM$

$$= \frac{1.5}{1} \times 0.6 \times 0.36 = 0.324\text{m} = 324\text{mm}$$

Depth of water required at the out let of the field$= \frac{324\text{mm}}{\text{Application Efficiency}} = \frac{324\text{mm}}{0.8} = 405\text{mm}$

iv. Irrigation interval$= \frac{324}{ET_c} = \frac{324}{5} = 64.8$ days adopt 64 days

v. Irrigation water required at head work $= \frac{405\text{mm}}{0.45 \times 0.35} = 2571.42$ mm

Example 2.15:

Table below gives the necessary data about the crop, duty of water and the area under each crop Commanded by a canal taking off from a storage reservoir. Taking a time factor for the canal to be 12/20, calculate the discharge required at the head of the canal. If the capacity factor is 0.8, determine the design discharge.

Crop	Base Period (days)	Area (ha)	Duty of water at the head of the canal (ha/cumecs)
Sugarcane	320	900	580
Overlap for sugarcane in hot weather	90	150	580
Wheat (winter)	120	750	1600
Rice (summer)	120	600	2000
Vegetable (hot weather)	120	320	600

Solution:

Here,

Discharge required for sugarcane = $\frac{900}{580}$ = 1.552 m^3/sec

Discharge required for overlapping sugarcane = $\frac{150}{580}$ = 0.259 m^3/sec

Discharge required for wheat = $\frac{750}{1600}$ = 0.469 m^3/sec

Discharge required for rice = $\frac{600}{2000}$ = 0.3 m^3/sec

Discharge required for vegetable = $\frac{320}{600}$ = 0.533 m^3/sec

Since sugarcane has a base period of 320 days, it will required water during winter, summer and hot weather. Thus

Discharge required in winter = 1.552+0.469 = 2.021 m^3/sec

Discharge required in summer = 1.552+0.3 = 1.852 m^3/sec

Discharge required in hot weather =1.552 + 0.259 + 0.533 = 2.344 m^3/sec

Out of three demands the maximum demand is 2.344 m^3/sec which is during the hot weather.

The time – factor is the ratio of the number of days the canal has actually run to the number of days the canal was supposed to run. In this case time factor is 12/20.

Discharge required at the head of the canal = 2.344 × 20/12= 3.907 m^3/sec

$$\text{Design discharge} = \frac{\text{Mean dischaege required}}{\text{Capacity factor}} = \frac{3.907}{0.8} = 4884 \text{ m}^3/\text{sec}$$

Example 2.16:

The base period, intensity of irrigation & duty of various crops under a canal irrigation system are given in the table below. Find the reservoir capacity if the canal losses are 18% and reservoir losses are 14%.

Crop	Base periods (days)	Duty at the field (ha/Cumecs)	Area under the crop (hectares)
Rice	120	850	3000
Wheat	120	1700	4500
Sugarcane	360	750	5400
Vegetables	120	650	1200
Cotton	200	1300	2200

Solution:

Calculations are done in the following table:

Crop	Base periods (days)	Duty at the field (ha/Cumecs)	Delta (m)	Area under the crop (hectares)	Volume of water required (ha-m)
Rice	120	850	1.22	3000	3660
Wheat	120	1700	0.61	4500	2745
Sugarcane	360	750	4.15	5400	22395
Vegetables	120	650	1.60	1200	1915
Cotton	200	1300	1.33	2200	2925
				Total =	33640

Total volume of water required by the crops = 33640 ha - m

Required capacity of the reservoirs = $33640/(0.82 \times 0.86)$

= 47702.78 ha - m

Example 2.17:

Determine the rate of application, if daily consumptive use of the crop is 6 mm and the canal may operates from 8 AM to 6 PM only. Available moisture for the given soil is 22% and maximum depth of root zone for the given crop is 1.2 m. Assume that only 50% of soil moisture is available to the crop. Application efficiency is 60%. Also calculate the required discharge if CCA is 350 ha.

Solution:

Given, Consumptive use = 6mm/day

Available water for soil= 22% = 220mm/m

Available water for crop = $220 \times 50\%$ = 110mm/m

Root depth of crop = 1.2m

Operation hrs of canal = 10 hrs (8 AM to 6 PM)

Application efficiency = 60%

Now,

Water available to crop with in root zone = 110 × 1.2 = 132 mm

Irrigation interval = $\frac{\text{water available to crop}}{\text{consumptive use}} = \frac{132}{6} = 22$ days

Required application= $\frac{\text{Number of days x consumptive use}}{\text{application efficiency}} = \frac{22 \times 6}{0.6}$ =220mm for every 22 days

Now, Rate of application = $\frac{\text{required application}}{\text{operation hrs}} = \frac{220}{10} = 22$ mm/hrs

Required discharge = $\frac{22 \times 10^{-3}\text{m}}{3600 \text{ sec}} \times 350 \times 10^4 \text{ m}^2 = 21.39 \text{ m}^3\text{/sec}$.

Example 2.18:

Write down the step wise procedure to determine the crop water requirement and Irrigation Requirement.

Solution:

Crop water requirements

Decide the future cropping pattern

↓

Calculate 15 days ET_0 values using (formulas or standard table)

↓

Take 15 days crop coefficient form standard table

↓

Calculatc ET crop mm/day or mm / half month

↓

Allow for land preparation (rice and wheat only)

↓

Allow for deep percolation (Rice only)

↓

Calculate evaporation form land preparation (rice only)

↓

Calculate total crop water requirements

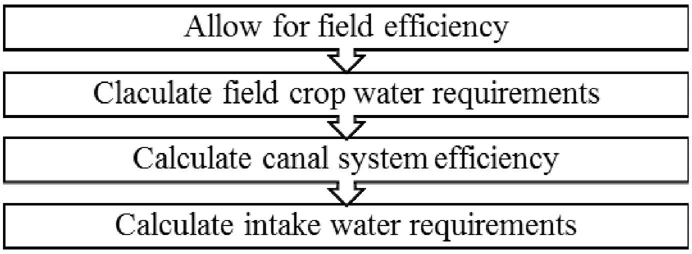
↓

Calculate effective rainfall

↓

Calculate net crop water requirements

Irrigation requirements



Example 2.19

Wheat is to be grown n field having a field capacity equal to 27% and the permanent wilting point 15.13%. Find the storage capacity is 80cm depth of soil, if the dry unit weight of the soil is 1.58m/cc. If the irrigation water s to be supplied when the average soil moisture falls to 18%, find the water depth required to be supplied to field if the field application efficiency is 80%. What is the amount of water needed at the canal outlet if the water lost in the water course and the field channel is 15% of the outlet discharged?

Solution:

(i) Maximum storage capacity = Available moisture

$$= \frac{\gamma d.d}{\gamma w} [FC - PWP]$$

$$= \frac{1.5 \times 80}{1} [0.27 - 0.13]$$

$$= 16.8 \text{ cm}$$

(ii) Depth of irrigation water = $\frac{1.5 \times 80}{1}$ [FC – OMC]

$$= \frac{1.5 \times 80}{1} [0.27 - 0.18]$$

(iii) Field irrigation requirement = $\frac{10.8}{0.8}$ = 13.5 cm

(iv) Amount of water needed at the canal outlet.

$$= \frac{13.5}{0.85} = 15.9 \text{ cm.}$$

Example 2.20:

The consumptive use requirements of a crop are 0.2 cm/day for days 1 to 15, 0.3 cm/day for days 16 to 40, 0.5 cm/day for days 41 to 50 and 0.1 cm/day for days 51 to 55. Effective rainfall of 3.5 cm, distributed uniformly during the 36th and 45th days (both inclusive) is predicted. Compute the total quantity of water (in cu. m) to be delivered to a 50 hectares area for the whole crop season with a pre - sowing requirement of 5 cm of water.

Solution:

Water Requirements per hectare

(i) 1 to 15 days i.e. 15 days @ 0.2 cm/day $= \frac{15 \times 0.2}{100} \times 10^4 = 300 \text{ m}^3$

(ii) 16 to 40 days i.e. 25 days @ 0.3 cm/day $= \frac{25 \times 0.3}{100} \times 10^4 = 750 \text{ m}^3$

(iii) 41 to 50 days i.e. 10 days @ 0.5 cm/day $= \frac{10 \times 0.5}{100} \times 10^4 = 500 \text{ m}^3$

(iv) 51 to 55 days i.e. 5 days @ 0.1 cm/day $= \frac{5 \times 0.1}{100} \times 10^4 = 50 \text{ m}^3$

$$\text{Pre - sowing Requirement} = \frac{5}{100} \times 10^4 = 500 \text{ m}^3$$

$$\text{Total} = 2100 \text{ m}^3$$

Effective rainfall/ hectare during 36th and 45th day (i.e. 10 days)$= \frac{3.5}{100} \times 10^4 = 350 \text{ m}^3$

Net water quantity to be delivered per hectare = 2100 – 350 = 1750 m^3

Now, Total quantity of water to be delivered to 50 hectares = 50 × 1750 = 87500 m^3

Exercise

1. Define Delta, Duty, base period, and crop period. Establish a relationship between duty and delta.
2. Write down the steps for calculating irrigation requirement for Rice crop.
3. Explain all types of irrigation efficiencies and Project efficiency.
4. Write down the methods to calculate Crop water requirement using Penman's method.
5. Explain Operational water requirement for paddy crops.
6. Explain evaporation and seepage losses in a canal.
7. Explain steps of irrigation water requirement for rice and wheat.
8. Define Soil moisture irrigation relationship with neat sketches.
9. Explain depth and frequency of Irrigation.
10. Explain how to estimate the design discharge for canal.
11. Describe different calculation methods of reference crop evapo-transpiration.
12. What do you understand by crop coefficient? What are the factors that affect the value of crop coefficient?
13. Calculate the optimum soil moisture and permanent wilting point of the silty loam soil, for following data:

    Density of soil =1.38 gm/cm^3; Depth of the irrigation= 6.15 cm; Field capacity=22.6%; Depth of root cone= 95 cm; Available moisture holding capacity= 12.36 cm.
14. If FC = 35cm/m; PWP = 15cm/m; root depth = 90cm; soil density = 1.5 gm/cc; Etc = 5 mm/day; application efficiency = 60% and RAM = 80% of AMC, than calculate:

(i) Available moisture content

(ii) Readily available moisture content

(iii) Depth of irrigation at the outlet of the field

(iv) Irrigation interval

15. If FC= 80%; PWP = 35%; root depth = 75cm; soil density = 1.5 gm/cc; ET_c = 5 mm/day; application efficiency = 80% and RAM = 50% of AMC, than calculate:

(i) Available moisture content

(ii) Readily available moisture content

(iii) Depth of irrigation at the outlet of the field

(iv) Irrigation interval

16. The culturable Commanded area of a Water Course is 750 ha. Intensity of irrigation for wheat is 45% and that of Sugarcane is 20%. The duties of the crops at the head of water course are 1500 ha/Cumecs and 800 ha/Cumecs respectively. Find the discharge required at the head of the water course.

17. The transplantation of rice usually takes 15 days and Δ of water required is 60cm on field. Due to rain about 15cm demand is fulfilled. Taking 12% losses from the distributary head to water course head and 20% losses in water course, compute

- Duty of water at the head of the water course.
- Duty of water at the head of distributary.

18. A canal takes off from a reservoir. Data for irrigated crops are given in table below:

Crop	Base periods (days)	Area under the crop (hectares)	Duty at the head of the canal (ha/cumecs)
Rice (summer)	120	240	775
Wheat (winter)	130	180	900
Sugarcane(overlap)	300	125	650
Vegetables (summer)	90	60	1000
Vegetable (winter)	110	90	1200

(i) Compute discharge required at the head of canal. Assume time factor as 0.7

(ii) Compute gross storage capacity of the reservoir. Assume losses due to absorption and evaporation as 10%, carry over storage (due to irregular monsoon) as 7% and dead storage as 10% of gross storage.

19. A stream of 130 lps was diverted from a canal and 100 lps were delivered to the field. An area of 1.5 ha was irrigated in 7.5 hrs. Effective depth of root zone was 1.6 m. The runoff loss in the field was 410 m3. The depth of water penetration varied linearly from 1.65 m at the head end of the field to 1.25 m at the tail end. Available moisture holding capacity of the soil is 25 cm per m depth of soil. Determine water conveyance, water application, water storage and water distribution efficiencies. The irrigation was started at a moisture extraction level of 60% of the available moisture.

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CHAPTER 3

Canal Irrigation System

A conveyance subsystem for irrigation includes open channels through earth or rock formation, flumes constructed in partially excavated sections or above ground, pipe lines installed either below or above the ground surface, and tunnels drilled through high topographic obstructions. Irrigation conduits of a typical gravity project are usually open channels through earth or rock formations. These are called canal. A canal is defined as an artificial channel constructed on the ground to carry water from a river or another canal or a reservoir to the fields. Usually, canals have a trapezoidal cross-section. The slope of an irrigation canal is generally less than the ground slope in the head reaches of the canal and, hence, vertical falls have often to be constructed. Power houses may be constructed at these falls to generate power and, thus, irrigation canals can be used for power generation also.

3.1 Classification of Canals

Canals can be classified in many ways (Basak, 2007).

1. Based on nature of sources of supply

(i) Permanent canal

A permanent canal has continuous sources of water supply. It is well graded channel and is provided with permanent regulation and distribution works. Such canals are further classified as:

– **Perennial canal**

Perennial canals are those canals which get continuous supplies from the sources throughout the year. The sources of supplied from these canal is a perennial river. These canal can supply water for irrigation throughout the year during the crop seasons.

– **Non – perennial canal**

Non perennial canals are those canals which get their supplies only for a part of the year. The source of supply for these canals is evidently a non-perennial river. These canals can therefore supply water for irrigation only during some of the crop season.

(ii) Inundation canal

The canal which is excavated from the banks of the inundation river to carry water to the agricultural land in rainy season only when the river flows to its full capacity is known as inundation canal as shown in Figure 3.1(a). No regulator is provided at the head of such canal. The flow of water through the canal depends on the fluctuation of water level in the river. When the water level rises above the bed level the water starts flowing through the canal as in Figure 3.1(b). When water level falls below the bed level of canal, the flow of water through the canal stops as in Figure 3.1(c).

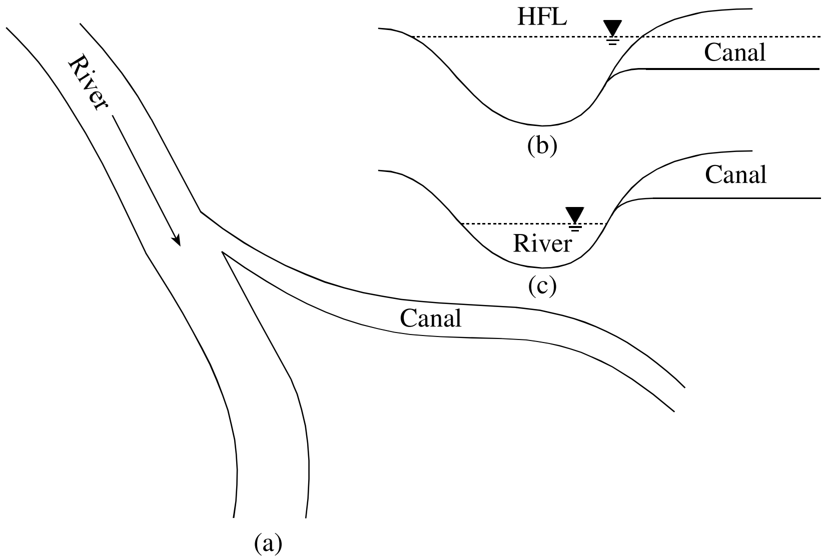


Figure 3.1: Nature of sources of supply

2. Based on their function (Purpose)

(i) Irrigation canal

The canal which is constructed to carry water from the sources to the agricultural land for the purpose of irrigation is known as irrigation canal.

(ii) Navigation canal

The canal which is constructed for the purpose of inland navigation is known as navigation canal. This type of canal is also utilized for irrigation.

(iii) Power canal

The canal which is constructed to supply water with very high force to the hydroelectric power station for the purpose of moving turbine to generate electric power is known as power canal or hydel canal.

(iv) Feeder canals

The canal which is constructed to feed another canal or river for the purpose of irrigation or navigation is known as feeder canal.

3. Based on carrying capacity

(i) Main canal

The main canal takes its supplies directly from the river through the head regulator and acts as a feeder canal supplying water to branch canals and major distributaries. Usually, direct irrigation is not carried out from the main canal.

(ii) Branch canal

Branch canal take their supplies from the main canal. Branch canals generally carry a discharge higher than 5 m^3/sec and acts as feeder canals for major and minor distributaries. Large branches are rarely used for direct irrigation. However, outlets are provided on smaller branches for direct irrigation.

(iii) Major distributary

Major distributary carry 0.25 m^3/sec to 5 m^3/sec discharge. These distributaries take their supplies generally from the branch canal and sometimes from the main canal. The distributaries feed either watercourses through outlets or minor distributaries.

(iv) Minor distributary

Minor distributaries are small canals which carry a discharge less than 0.25 m^3/sec and feed the water courses for irrigation. They generally take their supplies from major distributaries or branch canals and rarely form the main canals.

(v) Water courses

A water course is a small channel which takes its supplies from an irrigation channel (generally distributaries) through an outlet and carries water to the various parts of the area to be irrigated through the outlet.

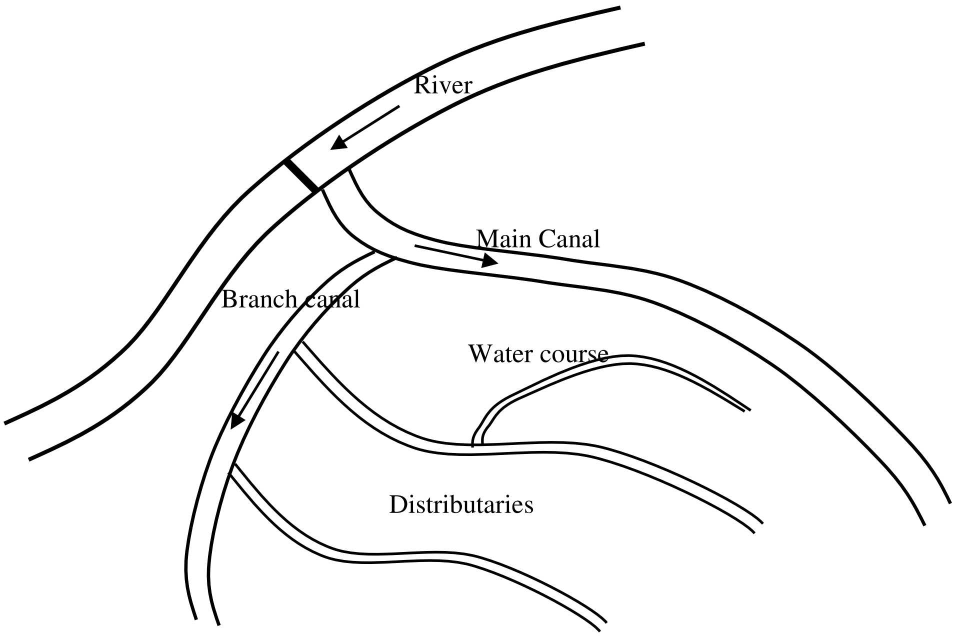


Figure 3.2: Based on carrying capacity

4. Based on alignment

Depending upon the alignment, the canals are designated as

a. Ridge or watershed canal

b. Contour canal

c. Side slope canal

(i) Ridge or water shed canal

The canal which is aligned along the ridge line (watershed line) is known as ridge canal or watershed canal. The advantage of this type of canal is that it can irrigate the areas on both sides. Again there is no possibility of crossing any natural drainage and hence no cross-drainage work is necessary.

(ii) Contour canal

The canal which is aligned approximately parallel to the contour lines is known as contour canal. This canal can irrigate the areas on one side only. This canal may cross natural drainage and hence cross- drainage works are necessary.

(iii) Side slope canal

The canal which is aligned approximately at right angle to the contour lines is known as side slope canal. It can irrigate the areas on one side only. Again, it doesn't cross any natural drainage and hence the cross drainage works are not necessary.

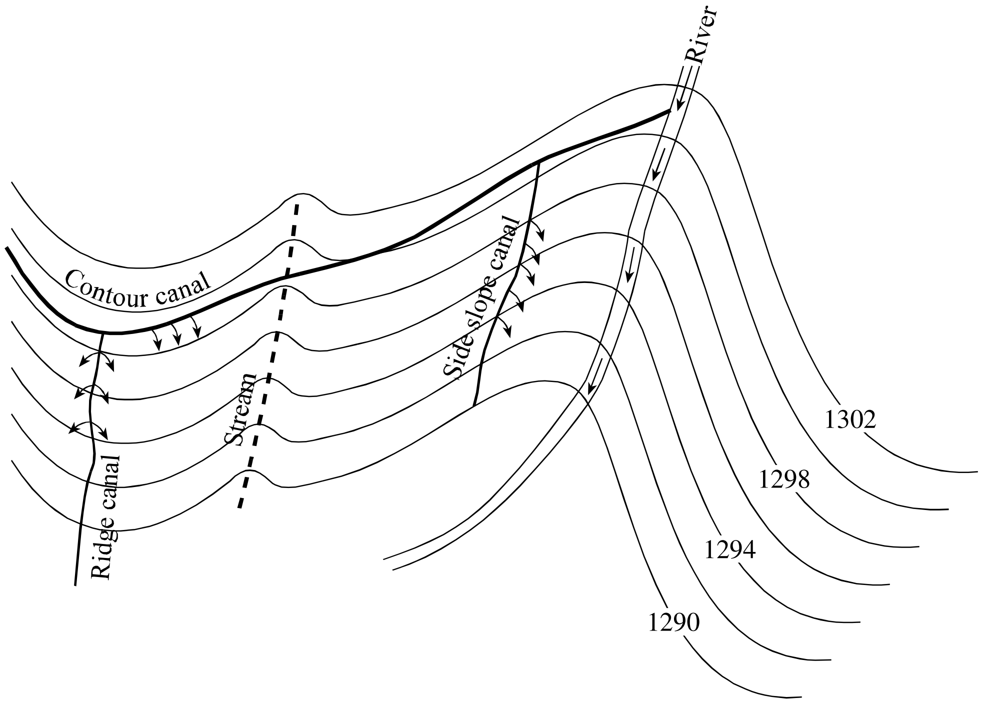


Figure 3.3: Side slope canal

5. Based on construction material

(i) Lined canal

Canals with an intention to reduce seepage are lined with different materials includes, concrete, shotcrete or plaster, brick or concrete tile, asphaltic concrete, boulder compacted earth and soil cement.

(ii) Unlined canals

Such canals are aligned and excavated either in alluvial soil or non - alluvial soils. An alluvial soil is one which is formed by the continuous silt deposition. The area of alluvial soil is comparatively even and has flat slope.

Rocky plain area formed due to disintegration of mountainous region is called non - alluvial area. Canals passing through such area are called non - alluvial canals.

6. Based on financial output

(i) Productive canal

It is one which earns revenue to the nation.

(ii) Protective canal

It is one which is constructed with an idea of saving a particular region from famine.

3.2 Component of Canal irrigation System

(i) Headwork

(ii) Canal network

1. Main canal
2. Branch canal
3. Distributaries
4. Minor
5. Water course

(iii) Canal Structures

1. Cross drainage structures
2. Canal fall or Drops
3. Cross regulators and head regulators
4. Outlets
5. Escapes for excess water
6. Village road bridges (VRB)
7. Via ducts (canal crossing)

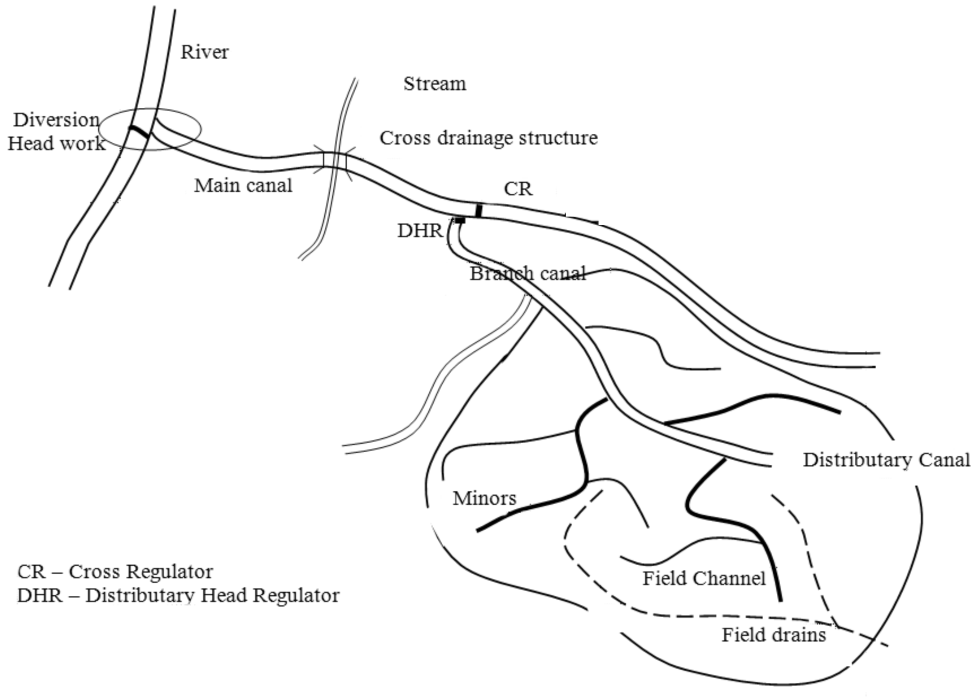


Figure 3.4: Component of Canal Irrigation System

3.3 Selection of Alignment of Canal

Desirable locations for irrigation canals on any gravity project, their cross-sectional designs and construction costs are governed mainly by topographic and geologic conditions along different routes of the cultivable lands. Main canals must convey water to the higher elevations of the cultivable area. Branch canals and distributaries convey water to different parts of the irrigable areas.

On projects where land slopes are relatively flat and uniform, it is advantageous to align channels on the watershed of the areas to be irrigated. The natural limits of commanded of such irrigation channels would be the drainages on either side of the channel. Aligning a canal (main, branch as well as distributary) on the watershed ensures gravity irrigation on both sides of the canal. Besides, the drainage flows away from the watershed and, hence, no drainage can cross a canal aligned on the watershed. Thus, a canal aligned on the watershed saves the cost of construction of cross-drainage structures. However, the main canal has to be taken off from a river which is the lowest point in the cross-section, and this canal must mount the watershed in as short a distance as possible. Ground slope in the head reaches of a canal is much higher than the required canal bed slope and, hence, the canal needs only a short distance to mount the watershed.

In general following points should be kept in mind while marking the tentative alignment of a canal:

- The alignment should not pass through the valuable lands, religious places, villages etc. to avoid unnecessary compensation and unwanted conflict.
- The alignment should be short as far as possible, but to make it short the alignment should not be taken through the area where irrigation is not at all possible.
- The alignment should be straight as far as possible.
- If the curve is unavoidable in the alignment, then it should be provided according to IS 5968-1970

  Some references are given in the following table:

Discharge (m^3/sec)	Radius (m)
80 – 100	1200 – 1500
30 – 80	800 – 1000
15 – 30	400 – 600
5 – 15	100 – 150

- The alignment should cross the natural stream, drainage etc approximately at right angles. At the crossing point, the width of the drainage should be minimum and the banks should be well defined.
- The alignment should not involve heavy cutting or banking. It is preferable if balancing depth of cutting and banking may be achieved.
- The alignment along the ridge line or watershed line is very good as the watershed canal can irrigate the areas on both sides. Moreover, the cross drainage works may be avoided.
- The alignment should be such that the maximum area may be irrigated with minimum length of canal.
- The alignment should not pass through the marshy land or water logged area, because the canal may collapse due to heavy moisture in the area.
- The alignment should not pass through sandy soil as the percolation loss will be more and the duty of canal will be less.

3.4 Alluvial and Non-alluvial canals

1. **Alluvial canals**

   The soil which is formed by transportation and deposition of silt through the agency of water, over a course of time is called the alluvial soil. The canals when excavated through such soils are called alluvial canals. Canal irrigation (direct irrigation using a weir or a barrage) is generally preferred in such areas, as compared to the storage irrigation (i.e by using a dam).

2. **Non alluvial canals**

   The soil which is formed by disintegration of rock formation is known as non – alluvial soil. It has an uneven topography, and hard foundations are generally available. The rivers, passing through such areas, have no tendency to shift their courses, and they do not pose much problems for designing irrigation structures on them. Canals, passing through such areas are called non – alluvial canals.

3.5 Canal Standards and Balancing Canal Depth

3.5.1 Canal Standards

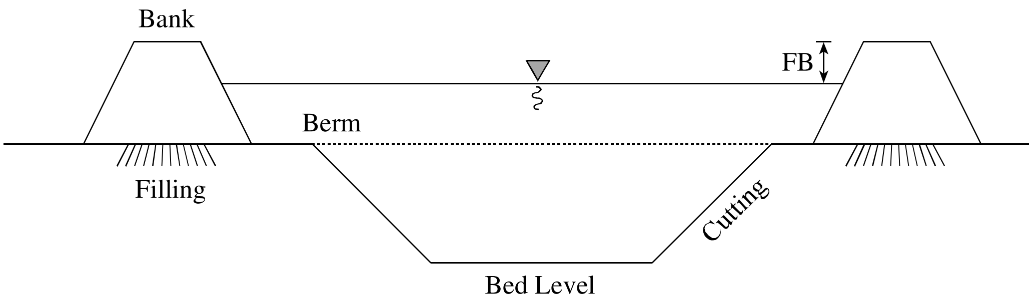


Figure 3.5: Elements of canal

A typical and most desired section of a canal is shown in Figure 3.5. This section is partly in cutting and partly in filling, and aims at balancing the quantity of earth work in excavation with that in filling.

Sometimes, when the ground level is above the top of the bank, the entire canal section will have to be built in cutting, and it is called **canal in cutting**.

And if the ground level is below the bed level of canal, the entire section will be in filling and it is called **canal in filling.**

1. **Side slopes**

   Side slopes in an unlined channel depend mainly on the nature of geological formations through which the channel is excavated. Side slopes in an unlined channel should be flatter than the angle of repose of saturated bank soil so that portions of side slopes will not slough into the channel. For similar conditions, the prevalent practice is to keep side slopes flatter for channels in filling than the side slopes for channels in cutting. However, there is no justification for this practice as a natural earth fill should not behave differently from an earth fill compacted sufficiently and having the same characteristics as those of the natural earth fill. Initially, flatter slopes are provided for reasons of stability. Later, with the deposition of fine sediments, the side slopes become steeper and attain a value of (1/2) H : 1V irrespective of the initial side slope provided. These steeper side slopes are stable and the design is usually based on these slopes. Side slopes for unlined channels in different types of soil, generally recommended are given in Table 3.1.

Table 3:1 Side slope for unlined channel

Type of soil	Side slopes (H:V)
Loose sand to average sandy soil	1.5:1 to 2:1 (in cutting) 2:1 to 3:1 (in filling)
Sandy loam and black cotton soil	1:1 to 1.5 :1 (in cutting) 2:1 (in filling)
Gravel	1:1 to 2:1
Hard soil	0.75 : 1 to 1.5 :1
Rock	0.25:1 to 0.5:1

2. Berm

A berm is a narrow horizontal strip of land between the inner toe of the bank and the top edge of cutting. Berms between water section and inner bank slopes are required along the channels where bank materials are susceptible to sloughing. Berms slope towards water section is to facilitate drainage.

Because of irregular changes in the ground level and regular changes in the channel bed level, the depth of cutting d1 and the depth of filling, $(d_2 - d_1)$ (Figure. 3.6) would vary. This will cause variation in the horizontal distance between the bed and top of the bank (i.e., between X and Y). If r_1: 1 is the side slope in cutting and r_2 : 1 is the inner side slope of embankment, then the berm width is kept equal to $(r_2 - r_1)\ d_1$. This will ensure that the horizontal distance between the bed and top of the bank (i.e., between X and Y) will remain constant. As can be seen from Figure. 3.6, this distance is equal to $r_1d_1 + (r_2 - r_1)\ d_1 + r_2\ (d_2 - d_1)$ which equals r_2d_2. Since the total height from the bed to top of the bank, d_2, remains practically constant, the horizontal distance between X and Y (i.e., r_2d_2) also remains constant if the berm width is equal to $(r_2 - r_1)d_1$.

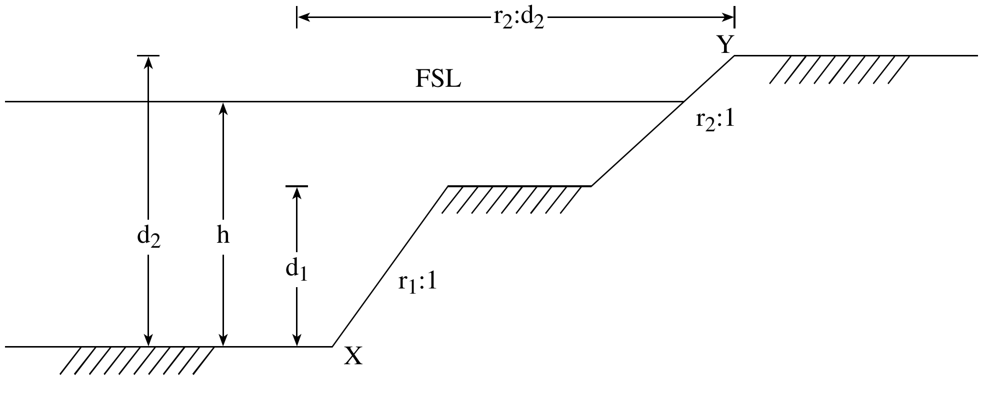


Figure 3.6: Berm

For usual side slopes of 1.5:1(H:V) (in filling) and 1:1(H:V) (in cutting), a berm of width equal to half the depth of cutting will make the horizontal distance between the bed and top of the bank (i.e., XY) equal to 1.5 times the height of the top of the bank with respect to the channel bed. As a result of silting on berm, an impervious lining is formed on the banks. This helps in reduction of seepage losses. In addition, the berm protects the banks against breaches and the eroding action of waves. These berms break the flow of rain water down the bank slope and, thus, prevent guttering. An additional berm may also be provided in channel sections which are in deep cutting.

3. Freeboard

Freeboard is the vertical distance from the water surface at full supply level to the top of bank. Freeboard provides the margin of safety against overtopping of the banks due to sudden rise in the water surface of a channel on account of improper operation of gates at the head regulator, accidents in operation, wave action, landslides, and inflow during heavy rainfall. The excessive growth of vegetation or accumulation of sediment deposits may also result in the gradual rise of water surface levels above the design levels. Design of channels should specify adequate freeboards to prevent overtopping of the banks during sudden rises in water surface. Adequate freeboard would depend on dimensions of the flow section, flow condition, bank material, method of construction of banks, and resulting damage due to failure of banks.

Freeboard in unlined channels vary from about 0.3 m in small channels to about 2 m in large channels. For channels of intermediate size, freeboard is sometimes estimated by adding 0.3 m to one quarter of the flow depth.

Table 3:2 Freeboard in irrigation canal (Singh, 1988)

Bed width (m)	Discharge (m^3/sec)	Freeboard (m)
Less than 1.0	-	0.30
1 to 1.5	-	0.35
Greater than 1.5	Less than 3.0	0.45
-	3 to 30	0.60
-	30 to 60	0.75
-	Greater than 60	0.90

USBR has proposed the following formula for the estimation of freeboard F (in meter) in canals.

$$F = \sqrt{CD}$$

Where, C is a constant varying from 0.45 (for discharge up to 0.07 m^3/sec) to 0.76 (for discharge greater than 85 m^3/sec) and D is the water depth in meter.

4. Canal Banks

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

A canal bank should be of such width that there is a minimum cover of 0.5 m above the saturation line. For large embankments of major canal projects, the position of the saturation line is determined as in case of earth dams and the stability of slope is computed using principles of soil mechanics. The saturation line for small embankments is drawn as a straight line from the point where full supply level meets the bank. The slope of the saturation line, i.e., the hydraulic gradient, may vary from 4H: 1V for relatively impermeable material (such as ordinary loam soil) to 10H: 1V for porous sand and gravel. For clayey soils, the hydraulic gradient may be steeper than 4(H): 1(V). If the bank section does not provide a minimum cover of 0.5 m above the saturation line, a counter berm, shown in Figure. 3.7 (b), is provided. Alternatively, the outer slope of the bank is flattened.

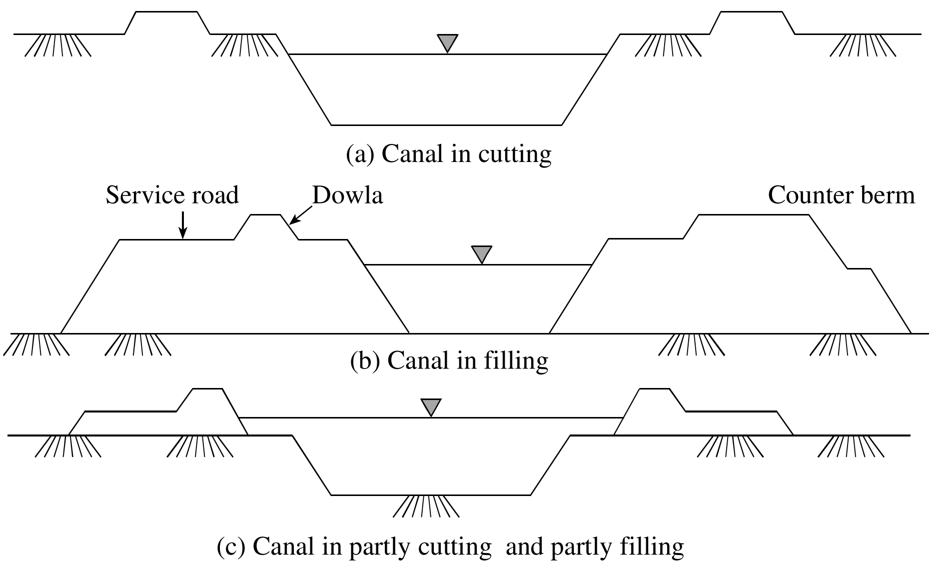


Figure 3.7: Typical cross section of canal

(i) Dowels/ Dowla

A Dowel or Dowla is provided on the side of a service road between the service road and channel. The top of the dowel is kept above the FSL in the channel the Dowel are provided as a measure of safety for automobile driven on the service road. They acts as a kerb on the side of the road way towards the canal.

(ii) Spoil bank

When the quantity of earth obtained from excavation or cutting is deposited near the cutting in the form of bank is known as spoil bank.

(iii) Borrow pits

When the earth work in filling exceeds excavated quantity at particular section borrow pit are required to make good requirement of filling.

3.5.2 Balancing Canal Depth

In constructing a canal section, if the quantity of excavated earth can be fully utilized for making the banks on both sides, then that canal section is known as economical section. The depth of cutting for that ideal condition is known as balancing depth. In this case, no borrow pit on spoil bank needs to be constructed. This condition may not occur in all the cases. It happens only when the canal section is partly in cutting and partly on banking. The cost of earth work will also be balanced.

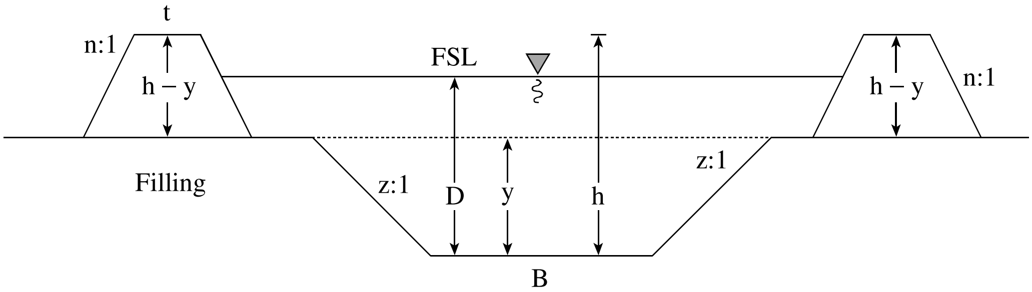


Figure 3.8: Balancing depth

Let h be the height of bank above the channel bed

D be the full supply depth

B be the bed width of channel

Y be the balancing depth

t be the top width of canal

n:1 be the side slope of the channel in filling

z:1 be the side slope of the channel in cutting.

Thus, area of cutting = $By + zy^2$

Area of filling = $2[\ t\ (h - y) + n\ (h - y)^2]$

Equating area of cutting and filling we get

$By + zy^2 = 2[\ t\ (h - y) + n\ (h - y)^2]$

$y^2\ (2n - z) - (B + 4nh + 2t)\ y + h(2t + 2nh) = 0$

Solving this equation and determine balancing depth.

The method of finding the balancing depth is described here.

Example 3.1

Find the balancing depth for canal section having the following data.

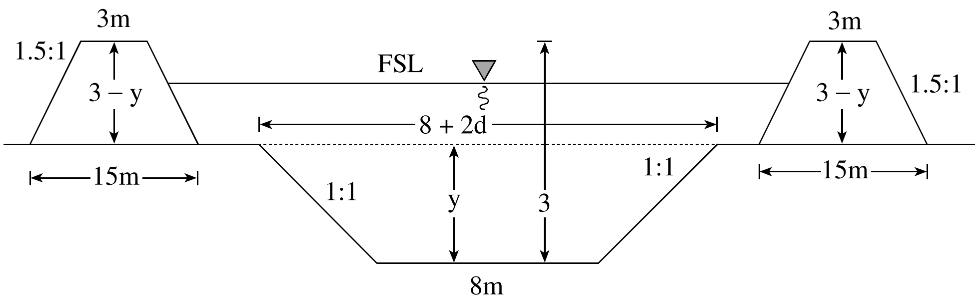
Base width of canal B = 8m

Side slope in cutting z = 1:1

Side slope in banking n = 1.5:1

Top width of bank t = 3m

Total height of canal = 3m



Solution:

From figure:

Equating the area of cutting and filling we get

$(B + zy)\ y = 2\ [t + n\ (h - y)]\ (h - y)$

Substituting the values and we get

$(8 + y) = 2\ [3 + 1.5\ (3 - y)]\ (3 - y)$

Solving we get, y= 1.92m

Hence the balancing depth is 1.92m.

3.6 Canal Distribution System

The distribution system of an irrigation scheme is defined as the network of canals (or pipe) and associated structure which conveys water from the sources (head works) to the cropped area in a controlled way. Many schemes have a drainage system, complementary to the distribution system for removal of surplus water, particularly excess rainfall and canal escape flows.

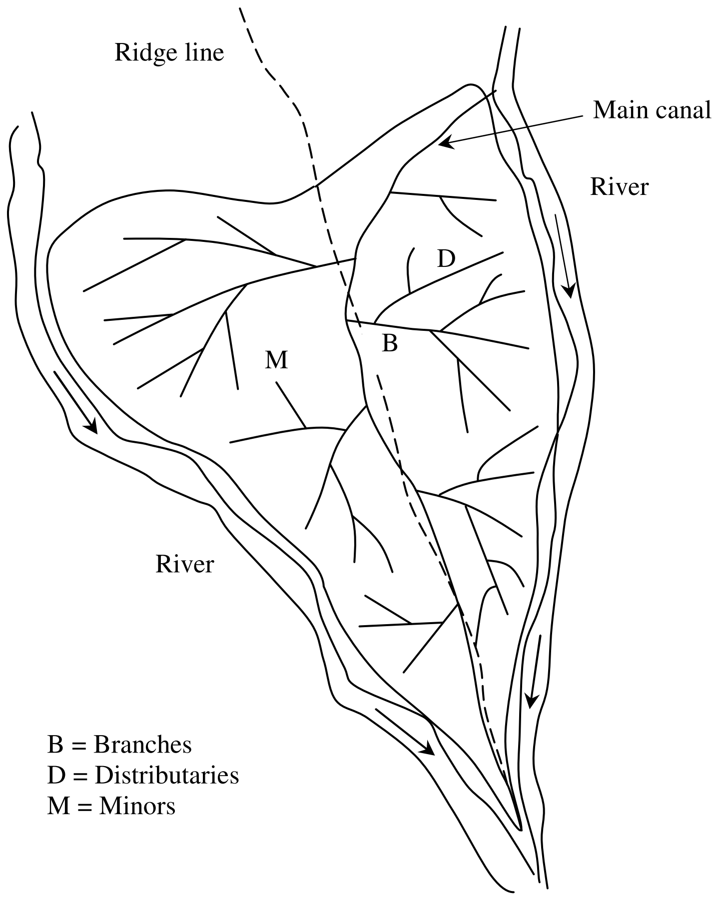


Figure 3.9: Canal distribution system

3.7 Methods of Water Distribution System

The objectives of the distribution system operation are to deliver the right amount of water to the right place, at the right time to meet crop needs. Ideally, water should be distributed equitably among farmers according to their needs. Irrigation systems in Nepal are generally of small to medium size, taking water from rivers origination in the non – snow fed mountains. This results in large variation in the supply, often with adequate supplies during the monsoon and insufficient water to feed the whole commanded area between November and May. This makes distribution difficult, and the design of the distribution system needs to be able to cope reasonably with large variations in the water available for delivery. In many cases, where all year cropping is required, there will be little alternative other than to supplement low river flows by groundwater abstraction, and to use this conjunctively with the available surface water.

Schemes fed from the larger rivers which have more reliable, perennial flows have other problems, principally siltation. Flood damage can be a problem on all types of schemes. While silt and flooding are best managed at the head of the system, the problem of low water availability has to be accommodated throughout.

There are many types of distribution system which differ according to the requirements and constraints placed on them. Some of the water distribution systems are as follows:

(i) On demand

(ii) Continuous flow

(iii) Scheduled

The advantages and disadvantages of some common types of distribution systems are summarized in Table below.

Types	Advantages	Disadvantages
On – demand	Farmers can take the water they want, when they want it.	Storage, high peak canal capacities and sophisticated structures usually mean high cost. Water is often wasted.
Continuous flow	Simple to operate, minimal number of control structures needed. Well suited to come crops, especially rice which grows in puddled conditions.	Farmers get the same flow regardless of what crop they are growing or the crop needs at the time. Flows are often small, so that losses are unacceptably high.
Scheduled	Farmers receive water according to a schedule which takes account of crop type and growth stage and thus meets (theoretical) crop needs.	Difficult to manage – requires a lot of calculation, canal operations and cooperation between farmers. Farmers perceived water needs are often different form calculated/ delivered needs, creating conflict.

Some of the principal factors affecting distribution system are described in table below. (M.8 Distribution systems, 1990)

Factor	Impact on distribution system
Scheme size	Affects number of levels in the canal hierarchy (typically two for small scheme)
Sources of water	Schemes fed form one (usually surface) source are a single entity with a canal hierarchy. Schemes fed from multiple sources (e.g. groundwater) normally comprise a number of discreet sub – systems.
Topography	Hill schemes often comprise a long canal feeding small pockets of land along its length, tending to give a “one – dimensional” effect. Terai schemes tend to more “two – dimensional”. Features such as natural channels, village boundaries, etc. will often determine the forms of the distribution units.
Demand / supply driven	The components of the system are determined by weather water is to be supplied continuously according to a schedule or “on demand”, in which case storage may need to be provided to cope with peaks and variations in demand, along with appropriate structures.
Flow variation	Large variations in flow availability will influence the rotational system chosen.
Management	If the management’s capability to operate the system is low, it may be preferable to provide semi – automated system (e.g. proportional distribution) so that gate operations objectives are partly compromised.
Charging	Charging farmers for water by volume may require the installation and operation of measurement devices.

Exercise 3:

1. What are the different types of canals?
2. How irrigation canals are classified? Explain in details.
3. Describe briefly the various considerations made in the alignment of an irrigation canal.
4. Explain with sketches the components of canal irrigation system.
5. How the canal is aligned in an irrigation system?
6. Define balancing depth of a canal with neat sketch.
7. An irrigation channel has a bottom width of 20m and side slopes of 1.5H: 1V in cutting and 2H: 1V in filling. The width of the crest of bank is 3m and its height above the ground level is 4.5m. Compute the balancing depth.
8. The following data refer to an irrigation canal.

   Bed width = 1.0m; side slopes = 2:1 (in filling) & 1:1 (in cutting)

   Top width of embankment on either side of canal = 3m

   Full supply depth = 5m; Free board = 1m

   Determine the balancing depth. Draw a neat x-section of the canal illustrating the various dimensions and levels.
9. Draw the typical cross sections of canal in (i) Filling (ii) Cutting (iii) partial cutting & filling.
10. Explain the canal distribution system and water distribution system.

✸✸✸

CHAPTER 4

Design of Canal

4.1 Design Capacity of Canals

The design discharge of the main canal off taking from a diversion headwork is fixed so as to achieve the optimum utilization of available water in the river during different parts of the year. The transit losses in the canal are estimated to determine the discharge that will be available at the canal outlets. (Arora, 2011)

Knowing the available discharge, the cropping pattern and duty of water, the culturable Commanded area can be worked out as follows:

1. Determine the water requirement of different crops for each month from the cropping pattern and the consumptive use.
2. Determine the outlet discharge required for each month for a culturable Commanded area.
3. Estimate the discharge available in the river for each month.

   Allowing for suitable transit losses, determine the discharge available at the outlets.
4. Determine the CCA which can be irrigated in each month from the available discharge found in step 3 and the required outlet discharge found in step 2.
5. The minimum of CCA found in step 4 is the designed CCA of the canal.
6. Determine the head discharge from the designed CCA and the maximum outlet discharge in step 2.

The above procedure is explained below with an example:

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Outlet discharge required for 1 lakh ha (m3/sec)	15.1	9.2	13.5	8.6	11.1	17.5	20.6	27.0	19.2	18.4	16.2	14.9
Available discharge (m3/sec)	65	59	56.5	54.2	65.2	108.2	201	310.2	210	130	110	85
C.C.A in Lakh ha	4.3	6.4	4.19	6.3	5.87	6.18	9.76	11.49	10.94	7.07	6.79	5.70

Maximum outlet discharge required (In Aug.) = 27.00 cumecs for 1 lakh ha.

Designed C.C.A. (In March) = 4.19 Lakh ha.

Assume transit losses = 30%

Head discharge of main canal= $\frac{27.0 \times 4.19}{0.7}$ = 161.61 cumecs.

4.2 Sediment Transport in Canals

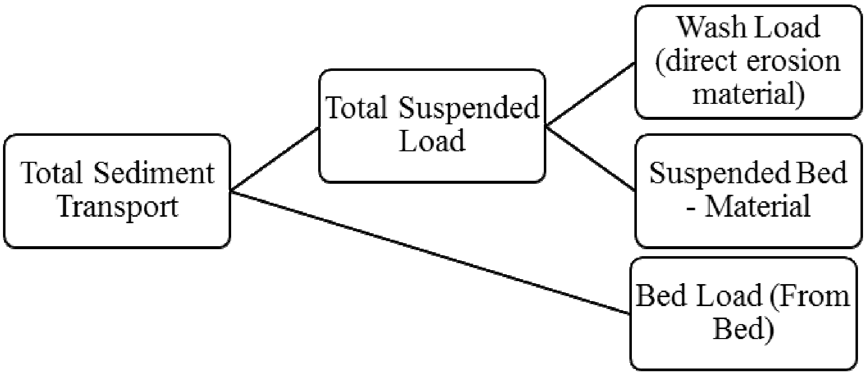
The transport of sediment in irrigation canals influences to a great extent the sustainability of an irrigation and drainage network. Unintentional or unwanted erosion and/or deposition of sediment in canals will not only increase the maintenance costs, but also lead to an unfair and inadequate distribution of irrigation water to the end users. Proper knowledge of the behavior and transport of sediment in these canals will help to plan efficient and reliable water delivery schedules to supply water at the required levels, to have a controlled deposition of sediments, to estimate and arrange the required maintenance activities, and to determine the type of desilting facilities and their efficiency, etc.

The study of sediment transport is mainly focused on the sediment and erosion processes in irrigation canal networks. In view of maintenance activities the head works should be designed in such a way that they prevent or limit the entrance of sediment into canals. In addition, the design of the canal network should be based upon the transport of all the sediment to the fields or to specific places in the canal system, where the deposited sediment can be removed at minimum cost. Sedimentation should be prevented in canals and near structures, as it will hamper and endanger a correct irrigation management, the main objectives of which are to deliver irrigation water in an adequate, reliable, fair and efficient way to all the farmers at the required water level, at the right time and at the proper rate. Inadequate management will result in low efficiency and unnecessary loss of the already scarce resource.

Irrigation canals are usually designed upon the assumption that the water flow is uniform and steady and that the canals are able to carry the water and sediments to the fields. The design supposes that an equilibrium situation exists where the sediments and water entering into the irrigation network will be transported to the fields without deposition or erosion. However, a perfectly uniform and steady flow is seldom found. In the operation of irrigation systems the flow is predominantly non-uniform, with varying discharges and very often with a constant water level at the regulation points where the water is supplied to the off – takes. The sediment transport capacity of the canals greatly depends on the flow conditions, which are variable. Although the water flow can be modelled with a high degree of accuracy, sediment transport behavior is only understood to a limited extent. The predictability of sediment transport equations and models in view of the quantity of sediment that needs to be removed is still rather poor. Computations of the effects of the non-equilibrium flow conditions on sediment transport are required to determine whether deposition and/or entrainment will occur and to assess the amount and distribution of the sediment deposition and/or entrainment along the canals. Mathematical modelling of sediment transport offers the possibility of estimating the distribution of sediment deposition or entrainment rates for a particular flow and a specific situation.

The main criterion for a canal design is the need to convey different amounts of water at a fixed level during the irrigation season in such a way that the irrigation requirements are met. Furthermore, the design must be compatible with the sediment load of a particular location in order to avoid silting and/or scouring of the canal. The water supply should meet the irrigation requirements and at the same time the supply should result in the least possible deposition in and/or scouring of the canals. The design process becomes more complicated when canals are unlined and pass through alluvial soils.

Sediments are fragmented material formed by the physical and chemical disintegration of rocks and they can be divided into cohesive and non-cohesive sediments. Non-cohesive sediments do not have physical-chemical interaction and their size and weight are important in view of their behavior. The total sediment transport in rivers and canals can be divided in suspended load and bed load as figure below.



4.3 Tractive Force Approach of Canal Design

When water flows in a channel, a force is developed that acts in the direction of flow on the channel perimeter. This force exerted by the flowing liquid on the channel perimeter is called tractive force or force drag or shear force. In uniform flow the tractive force is equal to the component of gravity force acting on the body of water parallel to the channel bottom (Durga P. Sangraula, 2017). Let us consider a uniform flow through an open channel with longitudinal slope so as shown in Figure 4.1.

Let's consider two sections 1 and 2. Let τ_o be the shear stress acting on the channel boundary and let W be the weight of the water inside the control volume.

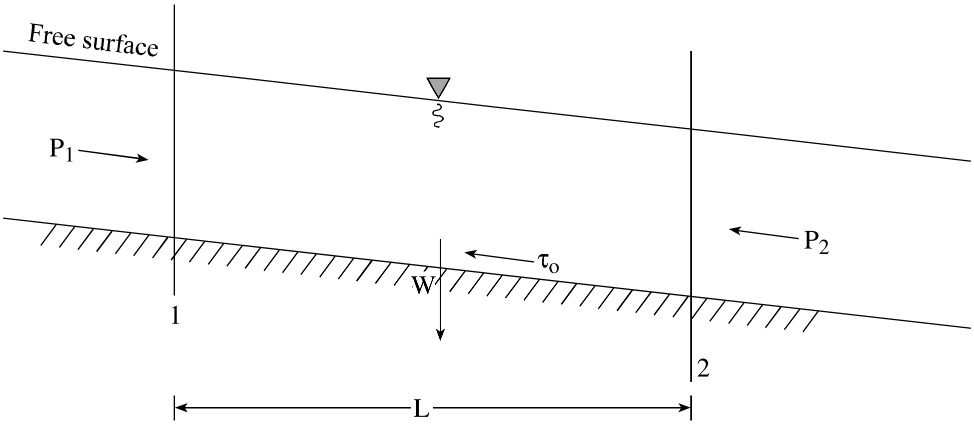


Figure 4.1: Forces Acting on a Flowing Fluid

Since the flow is uniform there is no acceleration, sum of forces acting in the flow direction must be zero (flow direction be *x*).

$$\sum F_x = 0$$

$P_1 - P_2 + W \sin\theta - \tau_0 \times \text{wetted area} = 0$

$P_1 = P_2$ (Equal and opposite hydrostatic pressure force).

$W \sin\theta = \tau_0 \times P \times L$

$$\tau_0 = \frac{W \sin\theta}{P \times L}$$

$$\tau_0 = \frac{\gamma\, A \times L \times S_0}{P \times L}$$ For small angle $\sin\theta = \tan\theta$ = slope of the bed.

$$\tau_0 = \gamma\, R\, S_0 \qquad \ldots 4.1$$

Equation 4.1 represents the equation for the unit tractive force or some time it is called normal shear stress. In a wide open channel, the hydraulic radius is equal to the depth of flow y: hence

$$\tau_0 = \gamma\, y\, S_0$$

Unit tractive force in canal except for wide rectangle is no uniformly distributed along the wetted perimeter. A typical distribution along with its pattern on unit tractive force acting on the bed and side slops of the trapezoidal canal is shown in Figure 4.2. As an approximation for a trapezoidal channel τ_0 at the bottom be assumed to equal to $\gamma\, y\, S_0$, and along the side slope to be equal to $0.75\, \gamma\, y\, S_0$. Figure 4.3 shows maximum unit tractive force on sides and bottom of various canal sections.

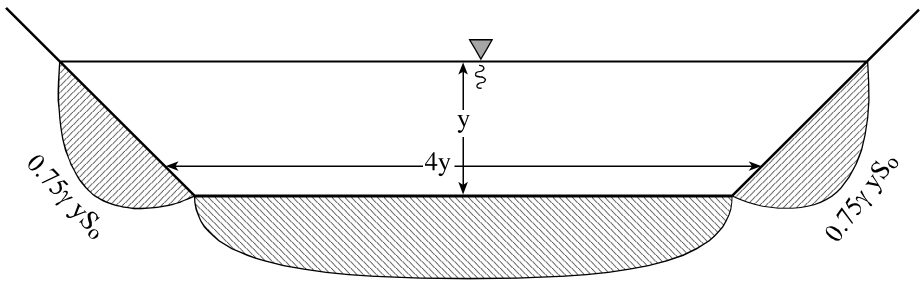


Figure 4.2: Tractive force distribution in a Trapezoidal canal Section

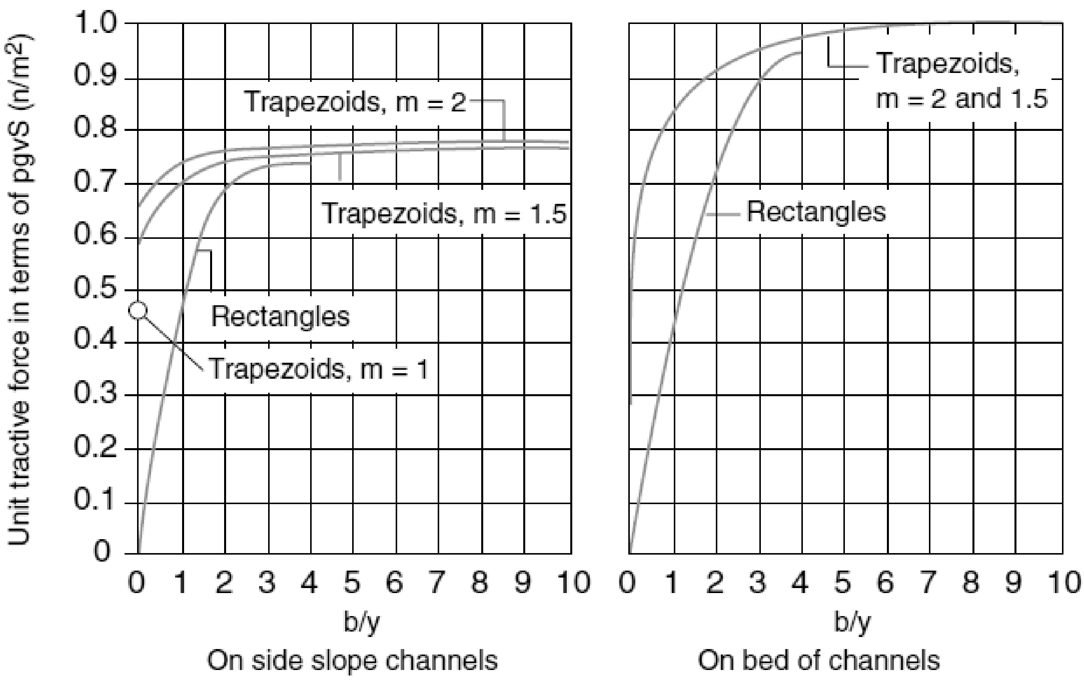


Figure 4.3: Maximum Unit Tractive Force on Sides and Bed Incipient Motion Condition

Equation 4.1 is developed on the assumption that no any particle on the bed and banks are moving. However, for the given configuration of channel section if we increase the shear stress the time comes when the particles will just start to move. The shear stress at which the channel materials (particles) just move from the stationary condition is called critical shear stress τ_c and fine motion is called incipient motion or motion is impending. In such condition of motion the particle diameter (generally d_{60}) on the bed is called critical particle diameter.

The particle size is usually denoted by d. For the identification of a particular grain size distribution its "percentage finer" value is used as sub index. The size is thus diving between equal weights of finer and coarser particles in a sample. It is some time erroneously termed as "mean diameter".

1. The Tractive Force Ratio

Scour on the perimeter of channel occurs when the particles on the perimeter are subjected to forces of sufficient magnitude to cause particle movement. Consider a trapezoidal channel section as shown in Figure 4.4. Let Ws be the submerged weight of the particle and θ be the angle of side slope of channel.

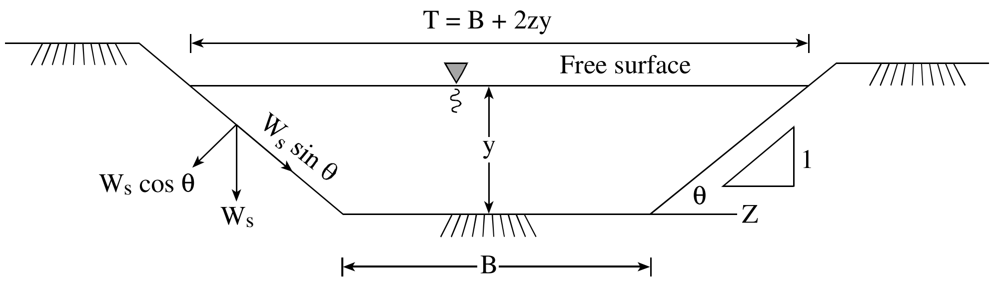


Figure 4.4: Trapezoidal channel section with forces

As shown in Figure 4.4, particles on the side slope of channel are less stable than those on the bed with similar condition of flow. This is because the component of weight of submerged particles (Ws sinθ) along the side slope tends to roll the material down the slope, thereby causing instability. If the resultant of two forces (force of flowing fluid and Ws sinθ) are more than the resisting force the erosion of canal slope may occur.

In the bottom of channel only the force of flowing fluid may scour the element. So bottom surface is more safety for scour than side slope. If the resistance force is less than that force scour may occur. So for designing mobile boundary channel the stability of side slope is the major concern.

Let's calculate these forces for channel at side slope and bed. Consider a particle of projected area "a "resting on the sloping side of a trapezoidal channel as shown in Figure 4.5. Let τs be the shear stress on the side, which will just cause the particle to move and let τs be the shear stress on the bed which will just cause the particle to move. Let Ws be the submerged weight of particle, θ be angle of side slope and φ be the angle of internal friction (angle of repose, property of soil forming the channel boundary)

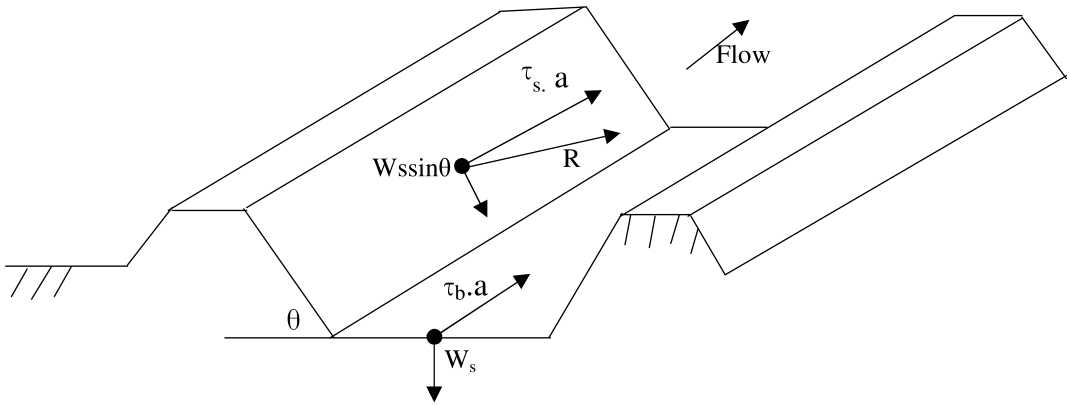


Figure 4.5: Analysis of forces acting on particle resting on sloping side

Resultant critical tractive force R on the side slope can be expresses as:

$$R = \sqrt{W_s^2 \sin^2\theta + \tau_s^2 a^2} \qquad \ldots 4.2$$

If the resultant is large enough particles will move. By the laws of frictional motion, for the incipient motion, the resistance to motion of the particles is equal to the force tending to cause the motion. The resistance to motion of the particles is equal to the normal force multiplied by the coefficient of friction (coefficient of friction is tanφ where φ - angle of repose or angle of internal friction and normal force is equal to Ws cosθ)

$$W_s \cos\theta \tan\varphi = \sqrt{W_s^2 \sin^2\theta + a^2 \tau_s^2}$$

$$W_s^2 \cos^2\theta \tan^2\varphi = W_s^2 \sin^2\theta + a^2 \tau_s^2$$

$$\tau_s = \frac{W_s}{a} \cos\theta \tan\varphi \sqrt{1 - \frac{\tan^2\theta}{\tan^2\varphi}} \qquad \ldots 4.3$$

Similarly, when motion of particle on the bed level is impending owing to the tractive force τ_b x a, the following equation may be written ($\theta = 0^0$).

$$\tau_b \times a = Ws \tan\varphi$$

$$\tau_b = \frac{Ws}{a} \tan\varphi \qquad \ldots 4.4$$

Ratio of $\frac{\tau_s}{\tau_b}$ is called tractive force ratio K and it is very important factor for design mobile boundary channel.

$$K = \frac{\tau_s}{\tau_b} = \cos\theta \sqrt{1 - \frac{\tan^2\theta}{\tan^2\Phi}} \qquad \ldots 4.5$$

$$K = \sqrt{1 - \frac{\sin^2\theta}{\sin^2\Phi}} \qquad \ldots 4.6$$

Equations 4-5 and 4-6 are mathematical identical. For given value of θ and φ we can calculate the tractive force ratio K, and this is important factor for designing mobile boundary channel. Figure 4.6 gives the angle of repose with particle size.

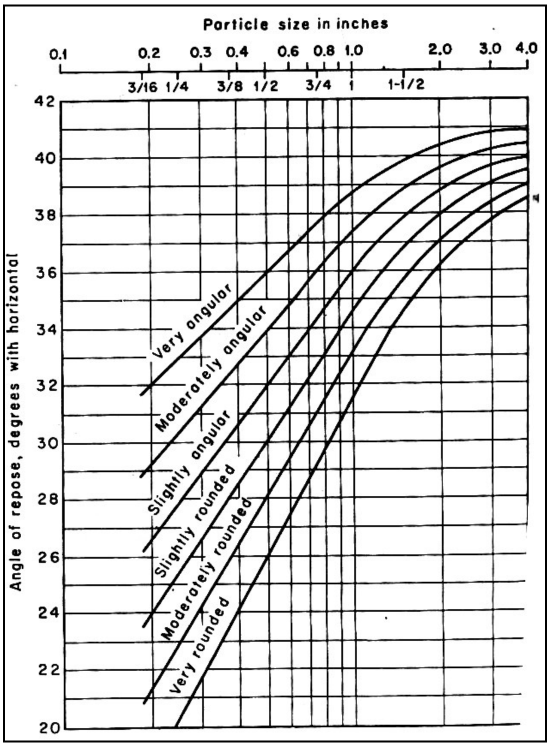


Figure 4.6: Angle of Repose for Non – Cohesive Material

The tractive force ration K is also known as the reduction factor for the critical stress on the channel side slopes. The value of K is always less than one but for horizontal slope K = 1 so that $\tau_s = \tau_b$. In the design of stable channel, the values of k(θ) normally controls the design values because the grain on the side slope tends to be more unstable than the grain of the bed. As mentioned above that critical stress is a function of material size and the sediment concentration. Furthermore, the critical shear stress at the channel sides is less than that at a level surface (bed), because the component of the weight along the side slope tends to roll the material down the slope, thereby causing instability. Figure 4.7 shows the variation of K with the angle of internal friction Φ and side slope of the channel α (in the text it is denoted by θ).

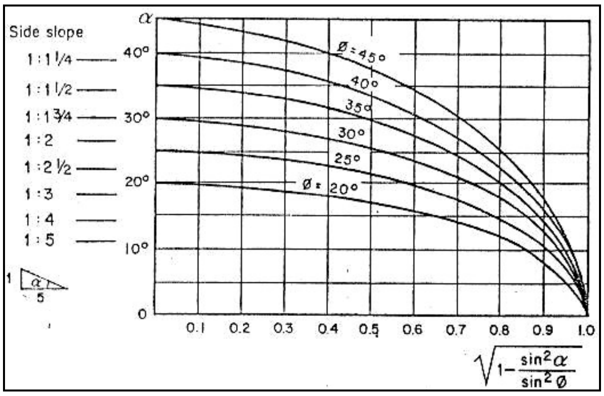


Figure 4.7: Reduction factor for critical shear stress for side slope

The maximum tractive stresses or permissible tractive stresses on the bed and sides $\tau_{max.b}$ and $\tau_{max.s}$ can be obtained for known values of B, y and z (bottom width, depth and side slope) from Figure 4.3. The design requires the determination of a section in which

$$\tau_{max.s} < \tau_s \quad ; \tau_{max.b} < \tau_b; \qquad \ldots\ 4.7$$

The design capacity of such channel can be obtained by using Manning's coefficient in which the rugosity coefficient is defined by Strickler's equation as

$$n = \frac{{d_{90}}^{1/6}}{24} \qquad \ldots\ 4.8$$

The design is usually carried out by using a trial and error process. The first step in the design of mobile boundary channels by this method is to select an approximate channel section i.e. B/y ration and z by experiences or from the design table given in the literature, collecting material forming the channel boundary and determining the coefficients (angle of repose). With these data, the designer investigates the section by applying tractive force analysis to ascertain probable stability by reaches and to determine the minimum section that appears stable.

One thing it must be noted that for trapezoidal channel as already mention above, the maximum tractive force on the sloping sides is always less than that on the bottom: hence, this force is the controlling value in the analysis. The section dimension of channel is carried out for the maximum tractive force on the sides; however, it is necessary to checking the proportioned dimensions for the maximum unit tractive force on the bottom.

The tractive force method of designing mobile boundary channel is more rational, than the regime theory, because it utilizes the laws governing sediment transport and resistance to flow. The regime theory is purely empirical

2. Shield's Tractive Force Theory

For particle purpose it is convenient to use Shield diagram for designing mobile boundary channel. Shields combined expressions for the destabilizing forces, drag and lift, against weight or friction as the stabilizing force into a general formula for the equilibrium of particles;

$$C_s = \frac{\tau_0}{(\gamma_s - \gamma_w)d} = \frac{\gamma\ R\ S_0}{(\rho_s - \rho_w)\ g\ d} \qquad \ldots\ 4.9$$

Here C_s is the dimension Shields number, ρ_s and ρ_w are density of dry particle and density of water and d is the diameter of the particle and generally consider as d_{50}. According to Shields, magnitude of the tractive force on the grain size depends on the fact whether the grain size outside or inside of the laminar sub-layer. If the thickness of the laminar sub-layer δ is more than the d50 i.e., grain is completely submerged in laminar sub-layer then tractive force on the grain depends on viscous or laminar flow (case when δ/d50 < 1). If $\delta/d_{50} > 1$, grain size remains outside the laminar sub-layer, then tractive force depends on turbulent flow outside laminar sub – layer.

Thus, the tractive force is directly related to d50/δ, i.e., $= \frac{\tau_0}{(\gamma_s - \gamma_w)\ d}$ ... 4.10

The equation of laminar sub-layer in boundary layer theory as given by Nikuradse's experimental data is (refer boundary layer for explanation):

$$\delta = 11.6 \frac{\mu}{v^*} \qquad \ldots\ 4.11$$

Where δ is the thickness of laminar sub-layer, μ is the kinematic viscosity of water and V* is the shear velocity equal to

$$V^* = \sqrt{\frac{\tau_0}{\rho}} \qquad \ldots 4.12$$

From equation 4.11 we can see that laminar sub-layer is directly proportional to $\frac{\mu}{V^*}$. Or we rewrite this as:

$$\frac{1}{\delta} \infty \frac{V^*}{\mu} \gg \frac{d}{\delta} \infty \frac{V^* d}{\mu} \qquad \ldots 4.13$$

Comparing equation 4.9 and 4.12 results:

$$\frac{\tau_0}{(\gamma_s - \gamma_w)d} \infty \frac{V^* d}{\nu}$$

Here $R_e^* = \frac{V^* \times d}{\mu}$ is called particle Reynolds number or roughness Reynolds number

So it indicates that $\frac{\tau_0}{(\gamma_s - \gamma_w)d} = f(R_e^*)$ ... 4.14

Plotting equation 4.13 on a log-log paper the well-known Shields curve (Figure 4.8) in sediment transport theory is obtained. This curve was found by experiments, using particles of different densities. It is therefore valid also for other materials than ordinary rock and other fluids than water.

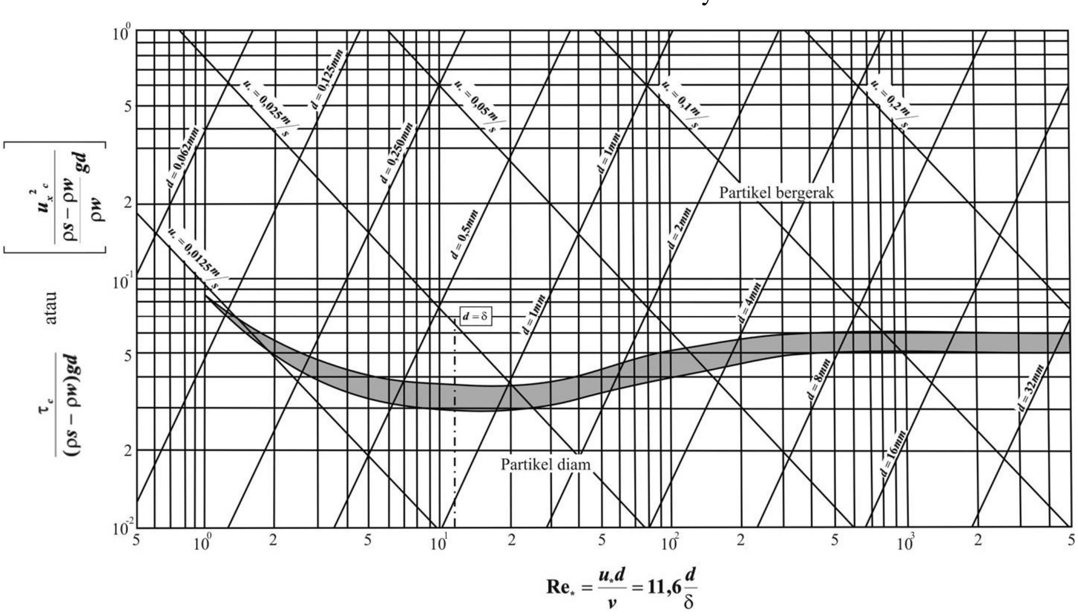


Figure 4.8: Shield Diagram for Incipient Motion Condition

When the value of particle Reynolds number Re* is less than 11.6, entrainment function depends on viscous force. When the value of particle Reynolds number Re* is more than 11.6, the curve slowly rises and becomes horizontal at about Re* ≈ 400. When Re*>400 than the value of

$$Cs = \frac{\tau_0}{(\gamma_s - \gamma_w) \times d} \approx 0.06$$

Becomes independent of Re* i.e., flow is fully turbulent. Value of C_s below the curve indicates stability against motion. Values on the curve indicate start of motion and are labeled critical Shields number C_s.

The corresponding shear stress is labeled as critical shear stress τ_c. The diagram is made for uniform sediments, but may be used for mixed sediments with good accuracy using d_{60} (60% finer) for d. For practical use C_s =0.06 can be used for grain diameters larger than 1 mm. For gravel Bed Rivers or rivers with coarse bed material the bed will not form. For gravel bed rivers or if the river bed is flat the flow resistance can be estimated based on Strickler's formula for the Manning's number n as (Lysne, Glocer, & Teskar, 2003):

$$n = \frac{d_{90}^{1/6}}{24} \text{ and } d_{90} = (1.75 \text{ to } 2.0)\ d_{60} \qquad \text{. . . 4.15}$$

Mittal and Swamee has worked out a general relation between τ_c and d which gives results within +5% of the values given by Shild's curve, for all values of d. The relation for water and soil of S_s = 2.65, is given by equation

$$\tau_c(N/m^2) = 0.155 + \frac{0.409\ d^2_{mm}}{\sqrt{1+0.177d^2_{mm}}} \qquad \text{. . . 4.16}$$

Example 4.1:

Water flows at depth of 0.6m in a wide stream having a bed slope of 1 in 2500. The median diameter of the sand bed is 1.0mm. Determine whether the soil grains are stationary or moving, and comment as to whether the stream bed is scouring or non – scouring.

Solution:

Since the given size of bed particles is 1.0 mm, which is less than 6mm, we cannot use shield's equation, since R_e* in this case will be less than 400. We will, therefore, use the general equation by mittal and swamee as follows:

$$\tau_c(N/m^2) = 0.155 + \frac{0.409\ d^2_{mm}}{\sqrt{1+0.177d^2_{mm}}}$$

$$= 0.155 + \frac{0.409 \times 1^2}{\sqrt{1+0.177 \times 1^2}} = 0.53\ N/m^2.$$

Also, we have

$$\tau_0 = \gamma\ R\ S_0$$

$$=9810 \times 0.6 \times 1/2500 = 2.35\ N/m^2 \text{ which is greater than } \tau_c\ ,$$

Since $\tau_0 > \tau_{c,}$ the soil grain will move and scouring and sediment transport will occur.

2. Relationship of grain diameter, hydraulic radius

For incipient motion condition i.e., the shear stress has reached its critical values τ_c, let's say particle diameter also as critical diameter dcr. In this situation the expression for Cs would be:

$$C_s = \frac{\tau_0}{(\gamma_s - \gamma_w)\times d} \geq 0.06 \Rightarrow \tau_0 \geq 0.06(\gamma_s - \gamma_w)d_{cr}$$

But $\tau_0 = \gamma_w RS_0$, where R being the hydraulic radius and S_0 is the bed slope.

$$d_{cr} \approx \frac{\gamma_w RS_0}{(\gamma_s - \gamma_w)\times 0.06} = \frac{R\,S_0}{0.06\left(\frac{\gamma_s}{\gamma_w} - 1\right)}$$

We know that $\frac{\gamma_s}{\gamma_w}$ is the specific gravity of the sediment ≈ 2.65

$$d_{cr} \approx \frac{RS_0}{(2.65-1)\times 0.06} = \frac{R\,S_0}{0.06\times 1.65} = \frac{RS_0}{0.099}$$

$$d_{cr} \approx 10\,R\,S_0$$

For safety propose dcr = 11 RS_o

Example 4.2

A canal is to be designed to carry a design discharge as 50 m^3/sec. The slope of the canal is 1:1000 and passes through medium with mean particles as 50mm. Assuming a trapezoidal section, determine the stable depth of canal assuming angle of repose of canal bed and side particles as 36^0.

Solution:

Given Q = 50 m^3/sec

S_0= 1:1000

d= 50mm

$\Phi=36^0$

Assume that side slope of the channel $\theta = 30^0$ (θ should be less than Φ)

Calculate tractive force ratio $K = \sqrt{1 - \frac{\sin^2\theta}{sin^2\Phi}}$

$$\frac{\tau_s}{\tau_b} = 0.525$$

Minimum shear stress required to remove the grain on the side slope is

$$\tau_s = 0.525\tau_b$$

This means that for stability;

$\tau_s \leq 0.525\tau_c$ … (i)

Actual shear stress on side slope

$\tau_s \leq 0.75\gamma R S_0$ …. (ii)

Comparing (i) and (ii)

$0.75\gamma R S_0 \leq 0.525\tau_c$

We know; $d_{cr} = 11RS_0$

$RS_0 = d_{cr}/11$

For critical; $\tau_c = \gamma_w RS_0 = \gamma_w d_{cr}/11$

$0.75\gamma R S_0 \leq 0.525 \gamma_w d_{cr}/11$

$R \leq 3.181\text{m}$

Assume y = 80% of R= 2.54m

Adopt y = 2.5m

Now; $A = By + zy^2 = 2.5(B+4.33)$

$P = B + 2y\sqrt{1+z^2} = B+10$

Manning's coefficient $n = \dfrac{d_{90}{}^{1/6}}{24}$

But we have, $d_{90} = (1.75 \text{ to } 2.0)\ d_{60}$

Assume $d_{90} = 1.75d_{60} = 87.5\text{mm}$

Now $n = \dfrac{0.0875^{1/6}}{24} = 0.0277$

From Manning's equation

$Q = \dfrac{1}{n} A R^{2/3} S_0{}^{1/2}$ …. (iii)

$R = \dfrac{A}{P} = \dfrac{2.5\ (B + 4.33)}{B + 10}$

Put the values in equation (iii) and solving we get

B = 6.083m

Adopt B = 6.1m

y =2.5m

Freeboard = 0.15 m

Example 4.3

Using tractive force approach, design a channel in alluvial soil for the following data:

(i) Discharge Q = 45 cumecs,

(ii) Bed slope = 1/4800

(iii) Manning's n = 0.0225,

(iv) Permissible tractive stress = 0.0035 kN/m^2

Side slope = 0.5:1

Solution:

$$R = \frac{\tau_c}{\gamma_w \times S} = \frac{0.0035}{9.81 \times 1/4800} = 1.713 \text{ m}$$

$$V = \frac{1}{n} R^{2/3} S^{1/2} = \frac{1}{00225} \times 1.713^{2/3} \times (1/4800)^{1/2} = 0.918 \text{ m/sec}$$

$$A = Q/V = 45 / 0.918 = 49 \text{ m}^2$$

$$P = A/R = 49/ 1.713 = 28.61 \text{ m}$$

$$P = 28.61 = B + 2.236 D$$

$$A = 49 = BD + 0.5 D^2$$

Solving these, we get D =1.94 m and B = 24.27 m

4.4 Design of Stable Canal

Stable channels refer to those channels, whose boundary (bed and side slope) are composed of hard material like rock cuts or artificial lining. Such channels can resist erosion satisfactorily. The design simply computes the dimensions of the channel by a uniform flow formula and then decides the final dimensions on the basis of hydraulic efficiency or empirical rule of best section, practicability and economy. Technique of minimizing the lined material, the minimum permissible velocity to avoid deposition of silt and debris on the channel bottom and side slope are the major criteria for designing of non – erodible channel. In designing stable canal, such factors as the maximum permissible velocity and the permissible tractive force are not the criteria to be considered.

The minimum permissible or non – silting velocity is the lowest velocity of the flow that doesn't allow sedimentation and growth of aquatic plants. Such velocity will prevent both sedimentation and vegetation growth. This velocity is very uncertain and its exact value cannot be easily determined.

The maximum permissible non - erosive water velocity in earthen canal should be such that on the one hand the canal bed does not erode and that on the other hand the water flows at a self-cleaning velocity (no deposition). A heavy clay soil will allow higher velocities without eroding than will a light sandy soil. A guide to permissible velocities for different soil is presented in Table 4.1 in winding canals, the maximum non - erosive velocities are lower than in straight canals.

Table 4.1 Maximum permissible velocity ranges (sources: FAO module 7)

Soil type	Maximum flow velocity (m/sec)
Sand	0.3 – 0.7
Sandy loam	0.5 – 0.7
Clayey loam	0.6 – 0.9
Clay	0.9 – 1.5
Gravel	0.9 – 1.5
Rock	1.2 – 1.8
Concrete	6.0
Steel lining	10.0

Lined canal can made a range of velocities, as erosion in not an issue. However, for easy management of water, the permissible velocities should be critical or sub - critical.

Bed width to water depth (B/D) ratio for trapezoidal canals

The recommended bed width and water depth (B/D) ratio for earthen trapezoidal canals are given in table below:

Table 4.2: Recommended B/D ratio

Water depth	B/D ratio
Small (D < 0.75 m)	1 (clay) - 2 (sand)
Medium (D = 0.75 - 1.5 m)	2 (cay) - 3 (sand)
Large (D > 1.5 m)	> 3

The bed width should be wide enough to allow easy cleaning. A bed width of 0.2 - 0.25 m is considered to be the minimum, as this still allows the cleaning of the canal with small tools such as a shovel. Lined trapezoidal canals could have similar B/D ratios as given table.

The design procedure is as follows:

1. Assume suitable permissible velocity (non silting and non scouring). If the amount of silt transport is low, it is recommended to assume higher side of maximum permissible velocity. This will also increase the discharge.
2. Longitudinal or bed slope can be fixed out considering the topography of land through which the channel has to pass.
3. Select Manning's co – efficient based on the material lining on the bed and side slope. It can be found in the literature.
4. Side slope of the channel z:1 (H:V) is to be assumed. It is recommended to select z =1.

5. If B is the bottom width and y is the full supply depth, then cross - section area can be calculated as (for trapezoidal channel section)

   $$A = (By + z\,y^2) \qquad \ldots \text{(i)}$$

6. Once discharge Q and velocity V are known, it means A is also known.
7. Using Manning's formula write the equation in terms of V and B as

   $$V = \frac{1}{n}\left(\frac{A}{P}\right)^{2/3} S^{1/2} \qquad \ldots \text{(ii)}$$

   Here V, A, z, n and S are known. Unknown values are B and y, which can be found by solving (i) and (ii).

8. Finally a free board is necessary to provide. Generally it is consider as 5% for low flow and 30% for high flow.

   Free board of the channel is the vertical distance between full supply level to the top of the bank. It prevents over toppling of bank when wave is produced due to the wind.

Example 4.4:

Design a rigid boundary irrigation channel laid on a slope of 0.0016 with discharge 9.1 m^3/sec. assume Manning's n = 0.015 with permissible velocity 1.3 m/sec.

Solution:

Given, Q= 9.1 m^3/sec, n = 0.015, S= 0.0016, V= 1.3 m/sec. and assuming side slope z = 0.5.

$$Q = AV$$

$$A = Q/V = 9.1/1.3 = 7.0\ m^2$$

Also $A = (By + z\,y^2) = 7\ m^2 \qquad \ldots \text{(i)}$

$$V = \frac{1}{n}\left(\frac{A}{B + 2y\sqrt{1+0.5^2}}\right)^{\frac{2}{3}} S^{1/2}$$

$$1.3 = \frac{1}{0.015}\left(\frac{7}{B + 2y\sqrt{1+0.5^2}}\right)^{\frac{2}{3}} 0.0016^{1/2}$$

$$0.34 = \frac{7}{(B + 2.236y)} \qquad \ldots \text{(ii)}$$

Solving (i) and (ii)

y= 0.35m

b= 19.8m

Assume free board = 25% of y = 0.125m

Bottom width B = 19.8m

Total depth y = 0.5m

And side slope z : 1= 0.5 : 1

4.5 Design of Alluvial Canals (Kennedy's & Lacey's Theory)

As already mentioned above that an alluvial channel is defined as channel in which the flow transports sediment having the same characteristics as that of material in the channel boundary. Such a channel is said to stable if the sediment inflow into a channel reach is equal to the sediment outflow. This indicates that the cross sectional area as well as the longitudinal slope do not change due to the erosion or deposition. The basic of designing such ideal canal is that whatever silt has entered to settle at its canal head is kept in suspension and is not allowed to settle anywhere along its course.

Empirical approaches of designing alluvial channel were made by Kennedy (1895), Lindely (1919) and Lacey (1930 – 1946), Ranga Raju – Misri's simplified design of Kenney's theory (1979). Some of the methods of designing erodible channel with regime theory are explained herewith.

4.5.1 Kennedy's Theory

Robert Kenney was an engineer in Punjab irrigation department. He made extensive study in 22 irrigation canals and their distributaries in Upper Bari Doab area in Punjab, India and found that the non – scouring, non – silting velocity Vc is related to the depth of flow y as

$$Vc = 0.55\ y^{0.64}$$

Here y is the flow depth in m and the V_c is the critical velocity (non – silting and non – scouring) in m/ sec. Kennedy claimed that sediment size plays an important role and so he defined a critical velocity ratio (CVR).

$$CVR = \frac{V}{V_c}$$

$$V = CVR \times V_c$$

$$V = CVR \times 0.55\ y^{0.64}$$

Where V is the mean velocity of flow can be calculated by Manning's equation of Kutter's equation. Kennedy found that CVR > 1 (1.1 to 1.2) to sediment coarser than Upper Bari Doab and CVR < 1 (0.8 to 0.9) for sediment finer than his canals.

To design the channel by Kennedy's methods, the item which must be known are:

i. Discharge
ii. Rugosity coefficient n (either Manning's or Kutter)
iii. Critical velocity ratio
iv. Either bed slope or B/D ratio.

Limitations of Kennedy's theory

1. Limitations of Kutter's equation is also incorporated in Kennedy's method
2. No separate equation for slope has been suggested. The slope is decided according to the slope of the ground available. So a range of slope can be given and for every slope, a separate channel can be designed. It can be shown that for a given discharge, n and m as slope varies, b and y vary and so b/y also varies. It is found that larger is the slope, smaller the b/y ratio. Kennedy's theory does not indicate as to which channel is the best for a given discharge.
3. He consider only average regime
4. No consideration was made of silt grade, silt charge, silt concentration and bed load.
5. Kennedy's theory is strictly applicable to the region for which it was developed even after the introduction of critical velocity ratio. So several regional formula were developed.

Case I:

Given Q, n, CVR and S

The procedure of designing alluvial channel by this method is as follows:

Assume a trial value of flow depth y

1. Determine the critical velocity V_0 by using equation V_0 = CVR x 0.55 $y^{0.64}$
2. Determine area by dividing discharge by velocity $A = Q/V_0$
3. Determine the channel cross section i.e. wetted perimeter.
4. Finally compute the actual mean velocity V that will prevail in the channel of this cross section, by using Kutter's formula, Manning's formula etc.

   a. Kutter's equation

$$V = C\sqrt{R\,S_0}$$

$$C = \frac{\frac{1}{n} + 23 + \frac{0.00155}{S}}{1 + (23 + \frac{0.00155}{S})\frac{n}{\sqrt{R}}}$$

$$\text{Now, } V = \frac{\frac{1}{n} + 23 + \frac{0.00155}{S}}{1 + (23 + \frac{0.00155}{S})\frac{n}{\sqrt{R}}}\sqrt{RS}$$

   b. Mannings' equation

$$V = \frac{1}{n}R^{2/3}S^{1/2}$$

5. Repeat the calculation till the two values of velocities determine in step 2 and 5 are nearly the same.

Case II

Given Q, n, CVR and B/D

i. Calculate A interms of D as indicated below.

   Let, $B/D = x$ (which is given)

   $B = Dx$

   Since $A = BD + z\,D^2$

   $A = xD^2 + zD^2 = D^2(x + z)$

ii. Substitute the value of A from step 1 and the value of V from equation V_0 = 0.55 x CVR x $D^{0.64}$ in $Q = A \times V$.

   $Q = D^2(x + z)\,(0.55\ \text{CVR}\ D^{0.64})$

   Solve for D;

   In this equation Q, CVR , x and z are known. Hence D is determine.

iii. Knowing D, calculate B and R from following relation.

$$B = x\,D$$

$$R = \frac{BD + zD^2}{B + 2D\sqrt{1 + z^2}}$$

iv. Calculate velocity V from $V_0 = 0.55\ CVR\ x\ D^{0.64}$

v. Knowing V, R & n determine bed slope S from Kutters equation or Manning' equation.

$$V = \frac{\frac{1}{n} + 23 + \frac{0.00155}{S}}{1 + (23 + \frac{0.00155}{S})\frac{n}{\sqrt{R}}}\sqrt{RS}$$

or, $V = \frac{1}{n} R^{2/3} S^{1/2}$

Example 4.5:

Design an irrigation canal to carry 30 cumecs of discharge. The channel is to be laid at a slope of 1 in 5500. The critical velocity ratio for the soil is 1. Use Manning's rugosity coefficient as n= 0.0225, channel side slope is 1:2 (H:V).

Solution:

Given, Q = 30 cumecs

CVR = 1

n = 0.0225

s = 1:5500

H: V= 1:2 i.e. z: 1 = 0.5:1

Step 1: $V_0 = 0.55\ CVR\ y^{0.64}$

Step 2: $A = Q/V_0 = 30/V_0$

Step 3: $A = By + zy^2$

$P = B + 2y\sqrt{1 + z^2}$

R= A/P

$V = \frac{1}{n} R^{2/3} S^{1/2}$

Step 4: Compare V_0 to V

Y	V_0	A	B	P	R	V
2	0.857	35.005	16.502	20.974	1.669	0.843
1.5	0.713	42.075	27.3	30.65	1.372	0.74
1.8	0.801	37.453	19.907	23.931	1.565	$0.807 \approx V_0$

Hence adopt, Bed width of canal = 19.91m

Depth of canal = 1.8m

Free board = 0.2m

Example 4.6:

Design an irrigation canal to carry a discharge of 5 cumecs. Assume Kutter rugosity n = 0.0225, CVR = 1 and B/D= 3.24.

i. B/D = 3.24

$A = BD + z D^2$ assume z = 0.5

$A= D^2(3.24 +0.5) = 3.74D^2$

ii. $V_0 = 0.55\ CVR \times D^{0.64}$

Q = A.V

$5 \text{ cumecs} = 3.74\ D^2\ (0.55 \times 1 \times D^{0.64})$

D = 1.40m

iii. B = 3.24x 1.40 = 4.54m

$$R = \frac{BD + zD^2}{B + 2D\sqrt{1 + z^2}} = 0.956m$$

iv. $V_0 = 0.55\ CVR \text{ x } D^{0.64} = 0.682$ m/sec.

v. $$V = \frac{\frac{1}{n} + 23 + \frac{0.00155}{S}}{1 + (23 + \frac{0.00155}{S})\frac{n}{\sqrt{R}}}\sqrt{RS}$$

Where n = 0.0225; R =0.956 and, using calculator

s= 1/4000

Example 4.7

Design an irrigation channel to carry a discharge of 5 cumecs. Assume n = 0.0225 and critical velocity ratio m = 1. The channel has a bed slope of 0.2m per kilometer.

Solution:

Step 1: Assume a trial depth of D = 1.0m

Step 2: $V_0 = 0.55 \text{ x m x } D^{0.64} = 0.55$ m/sec

Step 3: Area A = Q/v = 5/0.55 = 9.09 m^2

Step 4: $A = BD + D^2/2$

or, $9.09 = B \times 1.0 + 1^2/2$

B = 8.59m

Step 5: Perimeter $P = B + D\sqrt{5} = 8.59 + 1.0\sqrt{5}$

R = A/P = 9.09/10.83 = 0.84m

Step 6: $$V = \frac{\frac{1}{n} + 23 + \frac{0.00155}{S}}{1 + (23 + \frac{0.00155}{S})\frac{n}{\sqrt{R}}}\sqrt{RS}$$ [Where R = 0.84; S= 0.2/1000; n = 0.0225]

Now v = 0.555m/sec

Step 7: Ratio of velocities found in steps (6) and (2) = 0.555/0.55 = 1.009 ≈1

Hence the assumed D is satisfactory.

Example 4.8

Determine the dimensions of the irrigation canal for the following data:

B/D ratio =3.7; n = 0.0225; m = 1.0; s= 1:4000. Side slope of the channel is ½ horizontal to 1 vertical. Also determine the discharge which will be flowing in the channel.

Solution:

Here,

B/D = 3.7

B = 3.7D

For a channel with side slopes ½ :1

$$R = \left(\frac{BD + \frac{D^2}{2}}{B + D\sqrt{5}}\right) = \left(\frac{3.7D^2 + 0.5D^2}{3.7D + D\sqrt{5}}\right) = 0.708D$$

From Kennedy's equation $V_0 = 0.55 \times m \times D^{0.64} = 0.55D^{0.64}$

From Kutter's formula $$V = \frac{\frac{1}{n} + 23 + \frac{0.00155}{S}}{1 + (23 + \frac{0.00155}{S})\frac{n}{\sqrt{R}}}\sqrt{RS}$$

$$V = \frac{\frac{1}{0.0225} + 23 + 0.00155 \times 4000}{1 + (23 + 0.00155 \times 4000)\frac{0.0225}{\sqrt{0.708D}}}\sqrt{\frac{0.708D}{4000}}$$

$$V = \frac{0.975D^{1/2}}{1 + 0.781D^{-1/2}}$$

Equating the two values of V, we get

$$0.55D^{0.64} = \frac{0.975D^{1/2}}{1 + 0.781D^{-1/2}}$$

On solving we get, D = 1.0m

B = 1 × 3.7 = 3.7m

V=0.55m/sec

A = 4.2 m^2

Q = A × V = 2.31 cumecs.

4.5.2 Lacey's Regime Theory

Better and modified method of designing alluvial channel by empirical method was developed by Lacey. His regime theory considered the dimensions of bed width, depth and shape attain a state of equilibrium called regime state. In other word, a regime channel may be defined as a stable channel whose width, depth and bed slope have undergoes modification by silting and scouring and are so adjusted that they have attained equilibrium. Lacey also differentiated between initial and final regime condition of channel.

The initial condition is attained shortly after it is put into operation after construction and the canal begins to adjust its bed slope either by silting or scouring although bed widths in snot altered. The canal then appears to have attained stability, but it is not actually the final state of stability and hence: it still represent the initial regime condition.

Eventually continuous action of flowing water overcomes the resistance of the banks and setup a condition such that the canal that the canal adjust its complete section, the final or true regime condition is attained. After analyzing the large number of data of stable irrigation canal in Indo – Gangetic plains, he obtained a relation which in its converted from F.P.S to SI or metric units is as follows

$$V = \sqrt{\frac{2}{5} f R} \qquad \text{. . . 4.17}$$

Where, V = mean velocity of flow in m/sec.

f = silt factor

R= hydraulic mean radius in m.

This was the first regime equation given by Lacey.

For determining the channel dimension second equation would be necessary giving either cross sectional area A or perimeter P. lacey on the basis of plotted data thus evolved a relationship between A and V which is as follows.

$$Af^2 = 140\ V^5 \qquad \text{. . . 4.18}$$

[Where A= cross sectional area in m^2]

Other Derived Equations

1. Silt factor – Grain size relation

According to Lacey the silt factor f is dependent on the mean particle size of the boundary material in the channel. The value of f may be determined by the following relation given by Lacey.

$$f = 1.76\sqrt{m_r} = 1.76\sqrt{d_{mm}}$$

d = mean particle size in mm = d_{50} in mm.

2. V – Q – f relation

A relation between V – Q – f may be obtained from equation 4.18

We have $Af^2 = 140\ V^5$

Multiplying this equation by V on both side

$$A\ V\ f^2 = 140\ V^6$$

From continuity equation

$$Q = AV$$

$$Qf^2 = 140\ V^6$$

i.e. $$\mathbf{V = \left(\frac{Q\,f^2}{140}\right)^{\frac{1}{6}}}$$

3. P – Q relation (Perimeter – Discharge Relation)

We have

$$V = \sqrt{\frac{2}{5} f R} \qquad \text{. . . (i)}$$

And $Q f^2 = 140 V^6$ . . . (ii)

Squaring both side of equation (i) by two times;

$$V^4 = \frac{4}{25} f^2 R^2$$

$$f^2 = \frac{25 V^4}{4R^2}$$

Now from (ii)

$$Q \times \frac{25 V^4}{4R^2} = 140 V^6$$

We know R=A/P

Now we get,

$$\frac{25 Q}{4(A/P)^2} = 140 V^2$$

$$P^2 Q = \frac{560 V^2 A^2}{25}$$

$$P^2 Q = 22.4 Q^2$$

$$\boldsymbol{P = 4.75\sqrt{Q}} \qquad \text{. . . 4.19}$$

Where P = perimeter in m.

Equation 4.19 gives a direct relation between P and Q. It indicates that for a stable channel the perimeter P depends only on discharge Q and involves no other channel characteristic. This equation is commonly used for determining the waterway in the design of several hydraulic structures. The power or index of Q doesn't vary but the coefficient 4.75 seems to vary 3.6 to 6.15 depending on the nature of the transported material.

4. Regime flow equation

After determining the dimensions of a regime channel the bed slope of the channel is required to be determined. Thus by plotting a large number of data lacey obtained the following flow equation.

$$V = 10.8\, R^{2/3} S^{1/3} \qquad \text{. . . 4.20}$$

Where S = Bed slope

Equation 4.20 is called general regime equation which doesn't involve co – efficient of rugosity, as it is considered that rugosity is implicit in R and S. This equation is of considerable practical importance in evaluating flood discharge of rivers. In order to determine the flood discharge of rivers usually Manning's or Kutter's equation is used in which the coefficient of rugosity N is assigned a value on the basis of experience. However, the value of N cannot be specified with a higher degree of accuracy since it depends upon a large number of factors. As such the determination of flood discharge by the use of Manning's or Kutter's equation is unreliable. At the time of floods river is temporarily in regime and hence equation 4.20 may be applied which would give quite accurate and reliable results.

From these equations for determining the bed sloe of a regime channel three equations may be obtained as indicated below.

5. Regime slope equation

(i) S – f – R relation

We have the equation $V = 10.8\ R^{2/3} S^{1/3}$

Cubing both side of equation

$$V^3 = 1260\ R^2\ S \qquad \text{. . . (i)}$$

Also we have another equation

$$V = \sqrt{\frac{2}{5} f\ R}$$

Again cubing this equation

$$V^3 = (2/5)^{3/2}\ f^{3/2}\ R^{3/2} \qquad \text{. . . (ii)}$$

Hence from equation (i) and (ii) we have

$$1260\ R^2 S = (2/5)^{3/2}\ f^{3/2}\ R^{3/2}$$

On solving,

$$\boldsymbol{S = \frac{0.0002\ f^{3/2}}{R^{1/2}}} \qquad \text{. . . 4.21}$$

(ii) S – f – Q relation

We have

$$V = \sqrt{\frac{2}{5} f\ R} \qquad \text{. . . (i)}$$

$$V = \left(\frac{Q\ f^2}{140}\right)^{1/6} \qquad \text{. . . (ii)}$$

Equating equation (i) and (ii)

$$\sqrt{\frac{2}{5} f\ R} = \left(\frac{Q\ f^2}{140}\right)^{1/6}$$

$$R^{2/3} = \frac{5}{2f} 1/2\ \left(\frac{Q\ f^2}{140}\right)^{1/6} \qquad \text{. . . (iii)}$$

Substituting the value of $R^{1/2}$ from equation (iii) in equation 4.21, we get

$$S = \frac{f^{3/2}}{4980}\left(\frac{8.96f}{Q}\right)^{\frac{1}{6}}$$

$$\boldsymbol{S = \frac{f^{5/3}}{3340\ Q^{1/6}}} \qquad \text{. . . 4.22}$$

S – f – q relation

We have $V^3 = 1260\, R^2 S$

This can be written as

$$S = \left(\frac{V^2}{R}\right)^{\frac{5}{3}} \frac{1}{1260\,(RV)^{1/3}} \qquad \text{. . . (i)}$$

Again squaring both side of equation $V = \sqrt{\frac{2}{5} f R}$

$$V^2 = \frac{2}{5} f R$$

$$\frac{V^2}{R} = \frac{2}{5} f \qquad \text{. . . (ii)}$$

Further if q is discharge per unit width of the channel, then assuming it to be a wide channel, we have

$$q = RV \qquad \text{. . . (iii)}$$

Introducing equation (ii) and (iii) in equation (i) we get

$$S = \left(\frac{2}{5} f\right)^{5/3} \frac{1}{1260\, q^{1/3}}$$

$$\boldsymbol{S = \frac{0.000178\, f^{5/3}}{q^{1/3}}} \qquad \text{. . . 4.23}$$

Summary of Lacey's Formulas

(i) $P = 4.75\sqrt{Q}$ . . . 4.24

(ii) $f = 1.76\sqrt{d_{mm}}$ . . . 4.25

(iii) $V = \left(\frac{f^2 Q}{140}\right)^{\frac{1}{6}}$ . . . 4.26

(iv) $R = \frac{5}{2}\frac{V^2}{f} = 0.48\left(\frac{Q}{f}\right)^{\frac{1}{3}}$ . . . 4.27

(v) $S = \frac{3 \times 10^{-4} f^{5/3}}{Q^{1/6}}$ . . . 4.28

Here P is the wetted perimeter in m, R is the hydraulic radius in m, Q is the discharge m^3/sec, d is the diameter of sediment particle and f is the silt factor which depended on the types and size of sediment.

Example 4.9

A channel section is to be designed for the following data.

Discharge Q = 5 cumecs.

Silt factor f = 1.

Side slope (H:V) = 0.5:1

Also determine the bed slope of channel.

Solution:

Step 1: Velocity $V = \left(\frac{f^2 Q}{140}\right)^{\frac{1}{6}} = 0.574$ m/sec

Step 2: Area A= Q/V = 8.71 m^2

Step 3: Perimeter $P = 4.75\sqrt{Q} = 10.62$m

Step 4: $A = BD + zD^2$

$8.71 = BD + 0.5\ D^2$ . . . (i)

$P = B + \sqrt{5}\ D$

$10.62 = B + \sqrt{5}\ D$ . . . (ii)

From (i) and (ii), we get

B = 8.44m

D = 0.98m

Step 5: R =A/P = 0.823

Let's check with $R = \frac{5}{2}\frac{V^2}{f} = 0.824\text{m} \approx 0.823$ m hence Ok.

Step 6: Bed slope $s = \frac{f^{5/3}}{3340\ Q^{1/6}} = \frac{1^{5/3}}{3340 \times 5^{1/6}} = \frac{1}{4368} \approx \frac{1}{4400}$

Hence, the channel has bed width B = 8.44m, D = 0.98m and bed slope S = 1:4400.

Example 4.10

Design a stable irrigation canal carrying a discharge of 50 m^3/sec, which passes through alluvium d_{mean} = 0.50mm. Draw sketch of the designed section.

Solution:

Step 1: $f = 1.76\sqrt{d_{mm}} = 1.244$

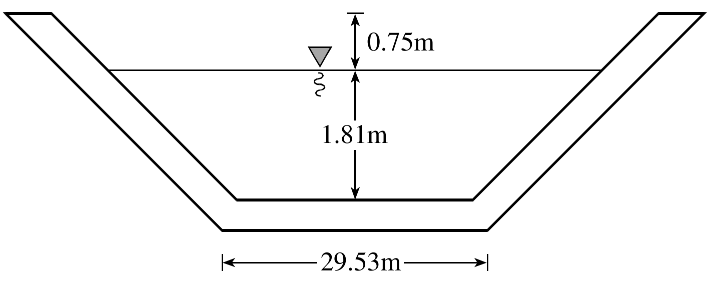
Step 2: Velocity $V = \left(\frac{f^2 Q}{140}\right)^{\frac{1}{6}} = 0.906$ m/sec

Step 3: Area A= Q/V = 50/0.906 = 55.187 m^3/sec.

Step 4: Perimeter $P = 4.75\sqrt{Q} = 33.587$m

Step 5: $A = BD + zD^2$ assume side slope z = 0.5

$55.187 = BD + 0.5\ D^2$ . . . (i)



$P = B + \sqrt{5}\,D$

$33.587 = B + \sqrt{5}\,D$ . . . (ii)

Form (i) and (ii), we get

B = 29.53m

D = 1.81m

Adopt free board = 0.75m

Overall depth = 2.56m

Step 6: R = A/P = 1.643m

Let's check with $R = \frac{5}{2}\frac{V^2}{f} = \frac{5}{2} \times \frac{0.906^2}{1.244} = 1.649\text{m} \approx 1.643\text{m}$. Hence Ok.

Step 7: Bed slope $s = \frac{f^{5/3}}{3340\,Q^{1/6}} = \frac{1.244^{5/3}}{3340 \times 50^{1/6}} = \frac{1}{4455.27} \approx \frac{1}{4500}$

Example 4.11

Design an irrigation channel in alluvial soil according to Lacey's silt theory for the following data.

Full supply discharge = 10 cumecs

Lacey's silt factor = 0.9

Side slopes of channel = 0.5:1 (H:V)

Solution:

Step 1: Velocity $V = \left(\frac{f^2 Q}{140}\right)^{\frac{1}{6}}$

$V = \left(\frac{0.9^2 \text{ x } 10}{140}\right)^{\frac{1}{6}} = 0.62\text{m/sec}$

Step 2: Area A = Q/V = 10/0.62 = 16.13 m^2

Step 3: Hydraulic mean radius $R = \frac{5}{2}\frac{V^2}{f} = \frac{5}{2} \text{ x } \frac{0.62^2}{0.9} = 1.07\text{m}$

Step 4: Perimeter P = A/R = 16.13/1.07 = 15.07m

Step 5: $P = B + \sqrt{5}\,D = 15.07$ . . . (i)

Step 6: $A = BD + 0.5D^2 = 16.13\text{m}^2$ . . . (ii)

Form (i) and (ii), we get

B = 12.27m; D = 1.25m

Step 7: Check $P = 4.75\sqrt{Q} = 15.02\text{m}$

$P = B + \sqrt{5}\,D = 15.07\text{m}$

These two values of P are nearly equal and hence the computations are correct.

Step 8: Bed slope $s = \frac{f^{5/3}}{3340\,Q^{1/6}} = \frac{0.9^{5/3}}{3340 \times 10^{1/6}} = 1/5844 \approx 1/5850$

Example 4.12

The slope of a channel in alluvium is 1/4000, Lacey's silt factor is 0.9 and side slopes are 0.5:1 (H: V). Find the channel section and maximum discharge which can be allowed to flow in it.

Solution:

We have the S – f – Q relation $S = \dfrac{f^{5/3}}{3340\ Q^{1/6}}$ Form this equation,

$Q = 1.03\ m^3/sec$

We also have S – f – R relation $S = \dfrac{f^{3/2}}{4980\ R^{1/2}}$

$R = 0.47m$

We also have; $P = 4.75\sqrt{Q} = 4.82m$

$A = PR = 4.82 \times 0.47 = 2.27m^2$

$A = BD + 0.5D^2 = 2.27$ . . . (i)

$P = B + \sqrt{5}\ D = 4.82$ . . . (ii)

Solving (i) and (ii), we get

$B = 3.48m$

$D = 0.60m$

4.6 Design of Lined Canals with Economic Analysis

Canal Lining

An irrigation channel is said to be lined when the bed and sides of the canal are protected by means of impervious or fairly impervious material of sufficient strength.

Necessity of lining

- To minimize seepage loss
- Prevention of water logging
- To increase discharge of canal section by increasing velocity
- To prevent erosion of bed and sides of the canal
- To reduce maintenance

Advantages

- Prevention of loss of water
- Prevention of water logging
- Low maintenance cost
- Less breaches
- Smaller cross sectional area
- Saving in canal structure

- Less silting and scouring
- Reduction of weed growth
- Increase value of land

Harmful salt from adjoining soil doesn't get dissolved in canal

Disadvantages

- High initial cost
- Difficult to repair
- Difficult to set outlets

Types of canal lining

1. **Hard surface lining**
    a. Cement concrete lining
    b. Shotcrete lining
    c. Pre - cast concrete lining
    d. Cement mortar lining
    e. Brick lining
    f. Stone blocks or undressed stone lining
    g. Asphalt concrete lining
2. **Earth type lining**
    a. **Soil - cement lining**
        - Cement 2 - 8 % added to fine soil
        - First dry mixed and water is added up to optimum moisture content
        - Placed, compacted and keep wet for few days
    b. **Clay puddle lining (30 cm thick)**
    c. **Compacted earth lining**
        - Soil graded with fine are thoroughly mixed
        - Water is added upto optimum moisture content and compact
    d. **Sodium carbonate lining**
        - Mixture prepared with soils by mixing 10% clay and 6% sodium carbonate

Economics of lining

Economics of lining is justified by B/C > 1

Where B = Annual Benefits

C = Annual Cost

Calculation of benefits

The major benefit then can be readily assessed in terms of money is from the saving of seepage water which would have been lost from unlined channel. This water when supplied to farmers will yield revenue.

Let q cumecs be the water saved in the lined reach and R_1 rupees be the cost of water per cumecs. Then the total saving = $q \times R_1$ rupees.

The other benefit is through a considerable reduction in the maintenance cost of a lined channel, compared to that of an unlined channel. Let p be the percentage saving in annual maintenance cost which is rupees R_2 for unlined channel. Then the saving in annual maintenance cost = $p \times R_2$ rupees.

Thus total benefit (B) per year = $q \times R_1 + p \times R_2$ rupees.

Saving in maintenance cost is usually taken to be around 40% so that p = 0.4

Besides the above benefits, there will be a considerable indirect benefits accrued from the prevention of water logging caused due to excess seepage, but as these are difficult to assess in terms of money, the minimum annual benefit (B) is considered as $B = q \times R_1 + p \times R_2$

Annual cost of extra expenditure on lining

Let the extra expenditure on canal lining be C rupees. If P_t is the total perimeter of lining, L is the length of lining and c is the cost of lining in rupees per m^2, Cost $C = c \times P_t \times L$. The extra expenditure is to be calculated by subtracting the saving in land cost, cost of drainage works, cost of bridges and earthwork etc. form the total cost of lining.

Let the life of lining be N years and *i* be the percentage rate of interest expressed as fraction per year. Then the capital recovery factor or annual cost of extra expenditure on lining equal to

$$= C\left(\frac{i\,(i+1)^N}{(1+i)^N - 1}\right)$$

Therefore benefit cost ratio B.C.R. is given by

$$\text{B.C.R.} = \frac{[q\,R_1 + p\,R_2]\,[(1+i)^N - 1]}{C \times i\,(1+i)^N} = \frac{B\,[(1+i)^N - 1]}{C\,i\,(1+i)^N}$$

Example 4.13

An unlined channel in alluvial soil has a seepage loss of 2.5 cumecs / million sq. m. of wetted perimeter. The channel has a wetted perimeter of 25 m and has maintenance cost of Rs. 1 per sq. m of wetted perimeter. There is a huge scarcity of water in the area and as such the canal is to be lined with cement concrete lining 12 mm thick so as to reduce seepage loss to 0.02 cumecs/ million sq. m. of wetted perimeter. The lined channel will have a wetted perimeter of 20 m. The extra cost of lining work out to be Rs. 100 per sq. m. If the average annual revenue per cumecs of water is Rs. 20 lakhs and percentage reduction in annual maintenance cost is 40%, decide whether it is economically feasible to provide canal lining. Assume the life of canal lining as 50 years and the interest rate is 6% per year.

Solution:

For the purpose of calculations assume 1 km length of channel

(i) Annual benefit

(a) Seepage

Area of wetted perimeter for unlined channel in 1 km = $25 \times 1000 = 25000$ sq. m.

Seepage loss = $25000 \times 2.5 \times 10^{-6} = 0.0625$ cumecs

Area of wetted perimeter for lined channel in 1 km = $20 \times 1000 = 20{,}000$ sq. m.

Seepage loss = $20000 \times 0.02 \times 10^{-6} = 0.0004$ cumecs

So, saving in water loss = 0.0625 - 0.0004 = 0.0621 cumecs

Annual revenue saved = Rs. $0.0621 \times 25 \times 10^{5}$ = Rs. 155250

(b) Maintenance

Annual maintenance cost of unlined channel = 1×25000 = Rs. 25000

Saving in maintenance cost = 0.4×25000 = Rs. 10000

Total annual benefit = Rs. 155250 + 10000 = Rs. 165250

(ii) Annual cost

Area of lining per km = $20 \times 1000 = 20000$ sq. m.

Cost of lining C = $100 \times 20000 = 2000000$

Interest rate = 6% per year. Hence $i = 0.06$, also N = 50 year.

Annual cost of lining or capital recovery factor $= C\left(\frac{i\,(i+1)^N}{(1+i)^N - 1}\right) =$

$$= 2000000\left(\frac{0.06\,(0.06+1)^N}{(1+0.06)^N - 1}\right) = \text{Rs. } 126888$$

Benefit cost ratio = $\frac{165250}{126888} = 1.302$ which is more than 1. The provision of canal lining is economically justifiable.

The lined canal can withstand much higher velocity than that of alluvial and non - alluvial channels. Design procedure is similar to non - alluvial channels.

The fundamental equations used for the designing of rigid boundary channel are the continuity equation and Manning's formula.

$$Q = AV$$

$$V = \frac{1}{n} R^{2/3} S^{1/2}$$

Where Q is the discharge, A is the area of flow, V is the mean velocity of flow, R is the hydraulic radius, n is the Manning's resistance coefficient (Manning's roughness or rugosity coefficient) and S is the longitudinal slope or it is also called a bed slope.

$$V = \frac{1}{n}\left(\frac{A}{P}\right)^{\frac{2}{3}} S^{1/2} \qquad \text{. . . 4.29}$$

The continuity equation can be written as,

$$Q = A\,\frac{1}{n}\left(\frac{A}{P}\right)^{\frac{2}{3}} S^{1/2} = \frac{Q\,n}{A^{5/3}\,S^{1/2}} = \frac{1}{P^{2/3}}$$

$$P = \frac{S^{3/4}}{(Q\,n)^{3/2}}\,A^{5/2} \qquad \text{. . . 4.30}$$

We can see from the Equation 4.30 that for a given discharge Q, Manning's n and bed slope S, the wetted perimeter P is minimum when the flow area A minimum. For minimum P and A, cost of excavation as well as lining used to prevent seepage and erosion will be minimum. Thus in the design of canal considering P to be minimum is termed as method of economic section. Further when discharge Q is maximum, P is minimum for a given value of n, S and A.

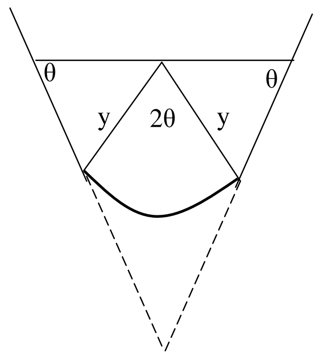
Most of the channels which are in practice are trapezoidal in shape, in lined channel side slope is taken 0.5:1 or 1: 1 (H: V). Thus, the method requires only finding best hydraulic sections i.e. the relationship of flow depth y with respect to bottom width B.

Theoretically, a semi - circular section is the best for open channel. For practical consideration, a trapezoidal or triangular section rounded corners is usually selected. Generally triangular section is selected for design discharge < 85 cumecs.

Generally, two types of channel sections are adopted.

(i) Triangular channel section

In order to increase R = A/P, the corners ra rounded and attempts are made to use deeper section by limiting depth, etc.



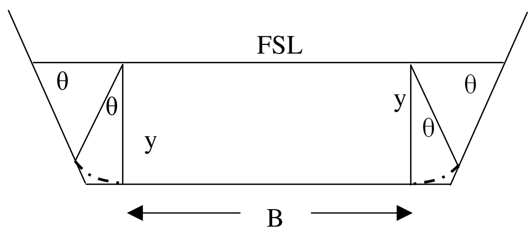
Let's central depth = radius of circle = y

Area $A = \pi y^2 \frac{\theta}{\pi} + 2 \frac{1}{2} y\,(y \cot\theta) = y^2\,(\theta + \cot\theta)$

Perimeter $P = 2\pi y\, \theta/\pi + 2y \cot\theta = 2y\,(\theta + \cot\theta)$

Hydraulic mean depth $= \frac{y^2\,(\theta + \cot\theta)}{2y\,(\theta + \cot\theta)} = y/2$ . . . 4.31

(ii) Trapezoidal



Area $= By + 2\,(\pi y^2\, \theta/2\pi) + 2 \frac{1}{2} y\,(y \cot\theta)$

$A = By + y^2\theta + y^2\cot\theta$ . . . 4.32

$A = y(B + y\theta + y \cot\theta)$

Perimeter $P = B + 2(2\pi y\, \theta/2\pi) + 2y \cot\theta$

$P = B + 2y\theta + 2y \cot\theta$ . . . 4.33

The channel is, then designed, according to Manning's formula. The value of rugosity coefficient (n) depends upon the roughness of the channel boundary, and is different for different kinds of linings. Different values of n for different types of lining materials are as shown is in table below.

Table 4.3: Values of Manning's (n) for Lined channels

Type of lining	Values of n for straight alignment
Cast in situ concrete trowel finish	0.015 – 0.018
Cement plastered masonry	0.012 – 0.015
P.C.C. Slab or tiles	0.018 – 0.020
Brick lining	0.018 – 0.020
Split Boulder lining	0.020 – 0.025
Round Boulder lining	0.025 – 0.035

Example 4.14:

Design an economical concrete lined trapezoidal channel to carry a discharge of 200 cumecs at a slope of 30 cm/km. The side slope of the channels is 1.5:1. The value of n may be taken as 0.017. Assume limiting velocity in the channel as 2m/sec.

Solution:

Given: $Q = 200\ m^3/sec$

$S= 0.3/1000 = 0.0003$

$V= 2m/sec$

$n= 0.017$

Step 1: Calculate R;

$$V = \frac{1}{n} R^{2/3} S^{1/2}$$

R= 2.75 m

Step 2: Calculate A & P

$A = Q/V = 200/2=100m2$

$P = A/R = 100/2.75 = 36.36m$

Now, $A = BD + D^2(\theta+\cot\theta)$

$\cot\theta = 1.5$

$\theta = \cot^{-1}(1.5) \times \pi/180= 0.588$ radian

Substituting the above value

$$100 = BD + D^2 (0.588 + 1.5)$$

$$100 = BD + D^2 \times 2.088 \qquad \text{. . . (i)}$$

$$P = B + 2D (\theta + \cot\theta)$$

Substituting above value we get

$$36.36 = B + 2D (\theta + \cot \theta)$$

$$36.36 = B + 4.176D \qquad \text{. . . (ii)}$$

From equation (i) and (ii)

$$B = 22.04 \text{ m}$$

$$D = 3.43\text{m}$$

Example 4.15:

Design a triangular shaped lined channel to carry a discharge of 15 cumecs. The available and accepted slope is 1 in 900. Assume suitable values of side slopes and good brickwork in lining.

Solution:

Given that: $Q = 15$ cumecs

$S_0 = 1/900$

Assume; side slope 1.25: 1 [H: V]

$n = 0.018$ for brick lining.

$\tan \theta = 1/1.25$

$\theta = 38.659 \times \pi/180 = 0.675$ rad.

Using $A = y^2 (\theta + \cot\theta)$

$P = 2y (\theta + \cot\theta)$

We get, $A = y^2 (0.675 + 1.25) = 1.925y^2$

$P = 2y (0.675 + 1.25) = 3.85y$

$R = A/P = y/2$

Using Manning's equation

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

$$15 = \frac{1}{0.018} (1.925y^2) (y/2)^{2/3} (1/900)^{1/2}$$

$y = 2.04$m

Adopt free board = 0.5

Adopt the overall depth of canal = 2.54m

Side slope = 1.25:1

Example 4.16:

Using tractive force approach, design a channel in alluvial soil for the following data:

(i) Discharge Q = 45 cumecs,

(ii) Bed slope = 1/4800

(iii) Manning's n = 0.0225,

(iv) Permissible tractive stress = 0.0035 kN/m^2

(v) Side slope = 0.5:1

Solution:

Here,

$$R = \frac{\tau_c}{\gamma_w \times S} = \frac{0.0035}{9.81 \times 1/4800} = 1.713 \text{ m}$$

$$V = \frac{1}{n} R^{2/3} S^{1/2} = \frac{1}{00225} \times 1.713^{2/3} \times (1/4800)^{1/2} = 0.918 \text{ m/sec}$$

$$A = Q/V = 45 / 0.918 = 49 \text{ m}^2$$

$$P = A/R = 49/ 1.713 = 28.61 \text{ m}$$

$$P = 28.61 = B + 2.236 D$$

$$A = 49 = BD + 0.5 D^2$$

Solving these, we get D =1.94 m and B = 24.27 m

Exercise 4:

1. Explain sediment transport and tractive force approach in canal design.
2. Explain semi theoretical approach of canal design.
3. Briefly describe the different methods of canal design.
4. Differentiate Lecey's and Kenney's silt theories.
5. An irrigation channel with protected sides is carrying a discharge of 2.7cumecs. The channel is constructed in course alluvium gravel of size 42mm and laid at a slope of 1 in 110. What is the minimum width to be provided? Compare critical shear using Shield's equation & Mittal & Swamee's equation.

6. A wide irrigation channel is laid at a slope of 1 in 4000. The size of the sand is 3.0mm. Water flows at a depth of 0.8m. Find whether the bed is moving or stationary. If the size of the sand is increased to 7mm, and the slope to 1in 1000 what will happen to the channel bed?
7. An irrigation channel is to be constructed in course alluvium gravel with 5mm particle size. The channel has to carry 2.9cumecs of discharge and the longitudinal slope is 0.01. The banks of the channel will be protected by grass against scouring. Find the minimum width of the channel.
8. Water flows at a depth of 0.7m in a wide stream having a bed slope of 1 in 2500. The diameter of the sand bed is 1.0mm. Determine whether the soil grains are stationary or moving. Also comment whether the stream bed is scouring or non-scouring.
9. An irrigation channel is to be constructed in course alluvium gravel with 4cm size. The channel has to carry 5 cumecs of discharge & the longitudinal slope is 0.04. The banks of the channel will be protected against scouring. Find the minimum width of the channel.
10. An irrigation channel is to be designed for the following data. Q = 35cumecs, S = 1 in 1400, d = 35mm, $\varphi = 37°$. The side slopes of the trapezoidal channel are unprotected.
11. A canal is to be designed to carry a discharge of 60cumecs. The bed slope is kept 1 in 1200. The soil is course alluvium having a grain size of 5cm. Assuming the canal is trapezoidal and unlined with unprotected banks, determine a suitable section for the canal. Assume $\varphi = 37°$.
12. An irrigation canal carries a discharge of 40m3/sec and is laid on a slope of 1 in 2500. If CVR = 1.2 and n = 0.025, design the channel.
13. Following dimensions were obtained while designing a trapezoidal canal.
14. Bed width = 21m, depth = 1.5m, Bed slope = 2.25 in 1000, Manning's n= 0.022, Find whether the chosen canal section is satisfactory.
15. A stable channel is to be designed for a discharge of 40 m3/sec and f =1.0. Calculate the dimensions of the channel using Lacey's regime equations. What would be the bed width of this channel if it is designed on the basis of Kennedy's theory? Adopt m = 1.0 and B/D ratio same as obtained from Lacey's equation.

✸✸✸

CHAPTER 5 Diversion Headworks

An irrigation channel takes its supplies from its source which can be either a river (in case of main canal) or a channel (in case of branch canals and distributaries). The structures constructed across a river source at the head of an off-taking main canal are termed as “canal headworks” or “headworks”. The headworks can be either diversion headworks or storage headworks.

Diversion headworks divert the required supply from the source channel to the off-taking channel. The water level in the source channel is raised to the required level so as to divert the required supplies into the off-taking channel. The diversion headworks should be capable of regulating the supplies into the off-taking channel. If required, it should be possible to divert all the supplies (at times of keen demand and low supplies) into the off-taking channel. The headworks must have an arrangement for controlling the sediment entry into the channel off-taking from a river. By raising the water level, the need of excavation in the head reaches of the off-taking channel is reduced and the commanded area can be served easily by flow irrigation. Storage headworks, besides fulfilling all the requirements of diversion headworks, store excess water when available and release it during periods when demand exceeds supplies

The site selection of a barrage/weir depends mainly on the location and elevation of the off-take canal, and a site must be selected where the river bed is comparatively narrow and relatively stable. The pondage requirement and interference with the existing structures such as bridges, urban development, valuable farmland, etc., must be considered, as well as available options to divert the flow during construction.

Function of Diversion Headworks

- It raises the water level on its upstream side
- It regulates the supply of water into canals
- It controls the entry of silt into canals
- It creates a small pond (not reservoir) on its upstream and provides some pondage.
- It helps in controlling the vagaries of the river.

5.1 Location of Headworks on River

Larger rivers, generally, have four stages, viz., the rocky, boulder, trough (or alluvial) and delta stages. Of these, the rocky and delta stages are generally unsuitable for sitting of headworks. Usually, the commanded area is away from the rocky stage, and it would, therefore, involve avoidable expenditure to construct a channel from headworks located in the rocky stage to its commanded area. In the delta stage, the irrigation requirements are generally less because of high ground water table and also the nature of the river at this stage poses other problems, like meandering and braiding.

The boulder and alluvial stages of a river are relatively more suitable sites for locating headworks. The choice between the boulder stage and the alluvial stage is mainly governed by the commanded area. If both stages are equally suitable for sitting the headworks from commanded area considerations, the selection of the site should be made such that it results in the most economical alternative. The following features of the two stages should be considered while selecting the site for headworks.

i. The initial cost of headworks in the boulder stage is generally smaller than that in the alluvial stage because of:
   - Local availability of stones,
   - Smaller width of river (requiring smaller length of weir),
   - Smaller scour depths which reduce the requirements of cutoffs and other protection works, and
   - Close proximity of higher banks which requires less extensive training works.

ii. An irrigation canal off-taking from a river in the boulder region will have a number of falls which may be utilized for generation of electricity. There is almost no scope for the generation of electricity in this manner in the alluvial reach of a river.

iii. If the existing irrigation demand is less but is likely to develop with the provision of irrigation facilities, it is desirable to divert the river water into an irrigation channel by constructing a temporary boulder bund across the river. This bund will be washed away every year during the floods and will be reconstructed every year. This will, no doubt, delay the Rabi crop irrigation, but it is worthwhile to use temporary bunds for a certain period; when the irrigation demand grows, permanent headworks may be constructed. In this manner, it would be possible to get returns proportional to expenditures incurred on the headworks. Construction of temporary bunds is generally not possible in the alluvial stage of the river.

iv. An irrigation channel off – taking in the boulder stage of a river will normally require a large number of cross-drainage structures. Because of the nature of the boulder region, there is always a strong subsoil flow in the river bed. This causes considerable loss of water and is of concern during the periods of short supply. Similarly, there will be considerable loss of water from the head reach of the off-taking channel. In alluvial reach of the river this loss of water is much less.

v. The regions close to the hills usually have a wet climate and grow good crops. The irrigation demand in the head reach of the channel off-taking in the boulder stage is, therefore, generally small. However, this demand would increase with the provision of irrigation facilities. In alluvial regions, the demand for irrigation is high right from the beginning.

For irrigation purposes, the site for headworks should result in a suitable canal alignment capable of serving its commanded area without much excavation. For siting the headworks, the river reach should, as far as possible, be straight and narrow and have well-defined and non-erodible high banks. In the case of a meandering river, the headworks should be located at the nodal point.

From sediment considerations, the off – taking channel should be located at the downstream end of the outside of a river bend so that it has the advantage of drawing less sediment. However, a curved reach would need costly protection works against the adverse effect of cross currents. Moreover, if canals take off from both the banks, the canal taking off from the inner bank draws relatively more sediment.

In order to ensure adequate supply to the off-taking canal at all times, the undersluice should be sited in the deep channel. A river reach with deep channels on both banks and shallow channel at the center is more suitable when canals take off from both sides.

Besides, the site must be accessible and suitable for making the river diversion and other related arrangements at a reasonable cost.

5.2 Component Parts of Weir and Barrage

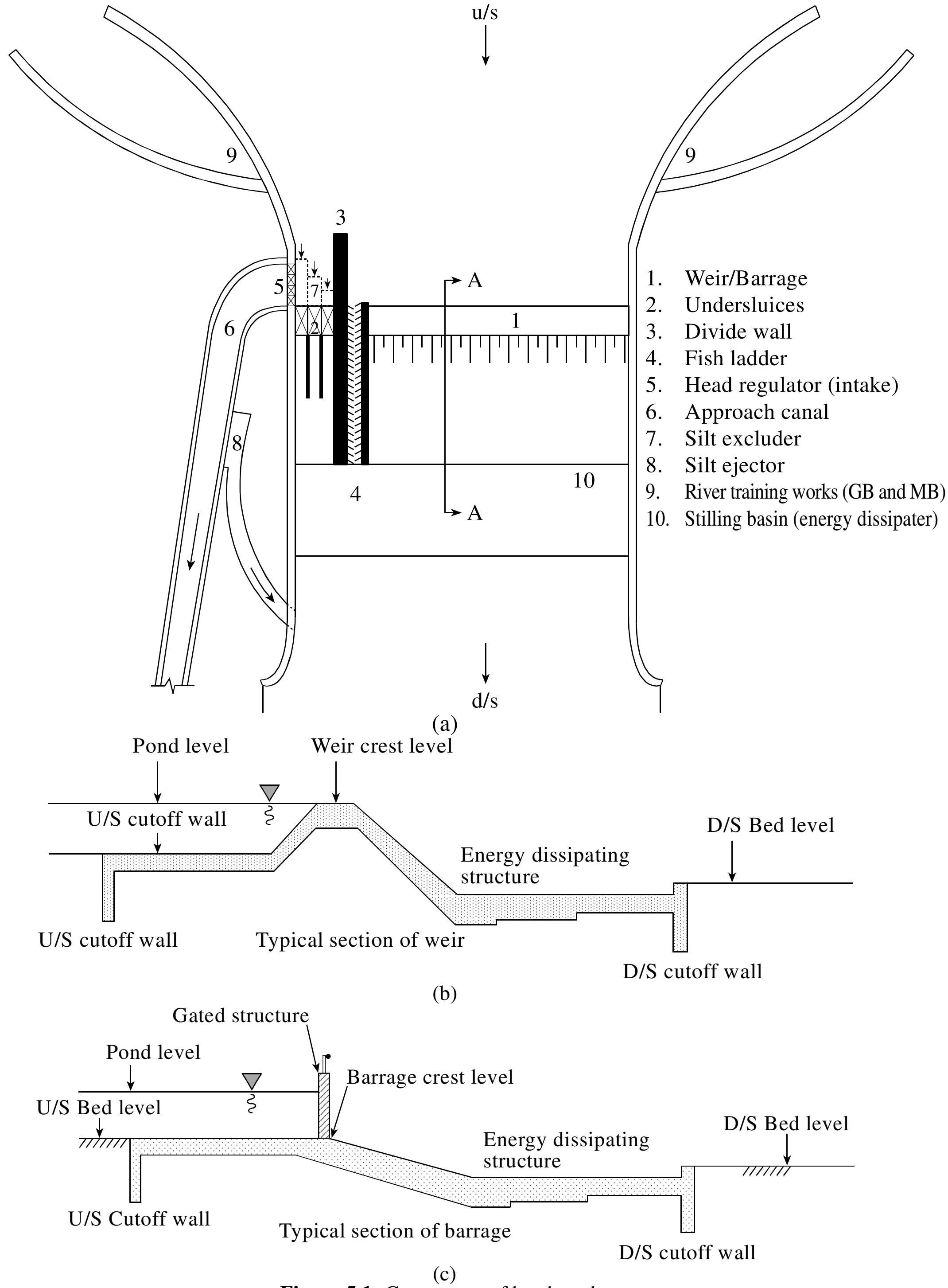


Figure 5.1: Component of head works

1. Weir and Barrage

Weirs and barrages are relatively low-level dams constructed across a river to raise the river level sufficiently or to divert the flow in full, or in part, into a supply canal or conduit for the purposes of irrigation, power generation, navigation, flood control, domestic and industrial uses, etc. Figure 5.1 These diversion structures usually provide a small storage capacity. In general, weirs (with or without gates) are bulkier than barrages, whereas barrages are always gate controlled. Barrages generally include canal regulators, low-level sluices to maintain a proper approach flow to the regulators, silt excluder tunnels to control silt entry into the canal and fish ladders for migratory fish movements. Weirs are also used to divert flash floods to the irrigated areas or for ground water recharging purposes. They are also sometimes used as flow measuring structures. Figure 5.1 (b) shows the section of a weir and Figure 5.1 (c) shows the section of a barrage.

The basic differences between weir and barrage are provided in Table 5.1.

Table 5.1: Differences between Weir and Barrage

	Weir	Barrage
Function	Weir is an impervious barrier which is constructed across a river to raise the water level on the upstream side is known as a weir. Here the water level is raised up to the required height and the surplus water is allowed to flow over the weir. Generally it is constructed across an inundation river.	When adjustable gates are installed over a weir to maintain the water surface at different levels at different times is known as a barrage. The water level is adjusted by operating the gates or shutters. The gates are placed at different tiers and these are operated by cables from the cabin. The gates are supported on piers at both ends. The distance between the pier to pier is known as Bay.
Sedimentation	Chances of silting on upstream is more	Silting may be controlled by judicial operation of gates.
Flow control	Low control on flow	Relatively high control on flow and water levels by operation of gates.
Back water effect	Afflux created is high due to relatively high weir crests	Due to low crest of the weirs (the ponding being done mostly by gate operation), the afflux during high floods is low. Since the gates may be lifted up fully, even above the high flood level.
Construction Cost	Low cost	High cost

2. Under- Sluices

These are controlled opening in the weir and barrage (even if weir doesn’t have gated structure) with crests at low level (river bed level). They are located on the same side near the off - taking canal. If two canals take off on the either side of river, it would be necessary to provide under - sluices on both sides.

Discharge capacity of the under sluices in provided higher of the following:

- Two times the maximum discharge in the off – take canal.
- 20% of the maximum flood discharge
- Maximum winter discharge.

3. Divide Wall

The divide wall is constructed parallel (or nearly parallel) to the canal head regulator. It separates the main weir bays from the bays of the under sluice as shown in Figure. 5.1. The wall extends on both sides of the weir. Extension of the divide wall towards the downstream of the weir avoids cross-flow in the immediate vicinity of the structure which, otherwise, may cause objectionable scour. The divide wall is usually extended up to the end of either the impervious floor or the loose apron on the downstream side. The divide wall serves the following purposes:

- It isolates the canal head regulator from the main river flow and creates a still pond of water in front of the canal head regulator. This results in deposition of sediment in the pocket and entry of relatively sediment - free water into the off taking canal. It also improves scouring of the undersluices by ensuring straight approach.
- It separates the weir floor from the floor of the undersluices which is at a lower level than the weir floor.
- If the main current has a tendency to move towards the bank opposite to the canal head regulator, the weir forces the water towards the canal head regulator. This causes cross-currents which may damage the weir. Under such adverse flow conditions, additional divide walls at equal intervals along the weir are provided to keep
- The cross-currents away from the weir.

When only one canal takes off from a river, the length of the divide wall should be half to two-thirds the length of canal regulator. When more than one canal takes off from the same bank, the divide wall should extend a little beyond the upstream end of the canal farthest from the weir. Some experimental studies have shown that a slight divergence of the divide wall from the regulator improves its efficiency. This divergence should not exceed 1 in 10. To reduce the scour at the nose of the divide wall, the nose end of the wall is given a slope of 3(V):1(H).

The divide wall is generally constructed as a strong masonry wall with a top width of about 1.5 to 2.25 m and checked for safety for the following two conditions:

- For low stage of the river, the water levels on the two sides of the walls are the same but the silt pressure is assumed to correspond to the sediment deposit up to full pond level on the pocket side.
- For the high stage of the river, the under sluice are discharging. At this condition, the water levels on the two sides are assumed to be different; the weir side level being higher by about 1.0 m.

It the river curvature is not favorable to sediment-free entry of water into the off-taking canal by inducing convex curvature opposite the head regulator, a second pocket of river sluice adjoining the under sluice improves flow conditions considerably. Such provision is useful in case of wide rivers to guide the river to flow centrally, minimizing cross-flow, and prevent shoal formation in the vicinity of the head regulator. The location and layout of the river sluice should be decided by model studies for satisfactory performance.

4. Fish Ladder

This is an artificial upstream fish passage, most commonly used for heads up to 20 m consisting of

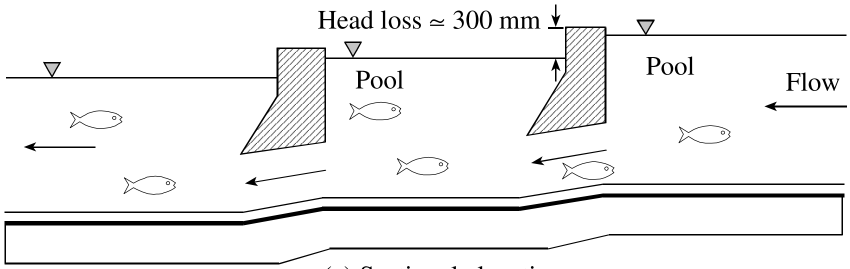
- A fish entrance
- A fish ladder proper and
- A fish exit. Sometimes an auxiliary (additional) water supply is also provided to attract fish to the entrance.

The fish ladder proper consists of a series of traverses (cross-walls) and pools circumventing an obstruction (such as a weir or dam) for the fish to migrate to the head waters in easy stages. This is achieved by creating a series of drops of around 300mm – 450mm between pools Figure. 5.2 on a gradient of around 1 in 8 to 1 in 15 (for high heads). Rest pools of a larger size (normally twice the size of an ordinary pool) are also provided after every 5 – 6 pools.

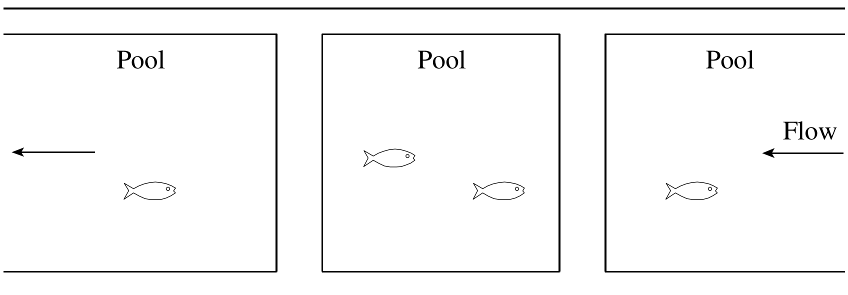
The actual arrangement of pools and traverses is chosen according to a particular obstruction; a low level weir or dam may need a fish pass of shallow gradient (corresponding to the surrounding gradient of the land) whereas a tightly folded pass may be necessary in case of a high weir or dam.

The fish pass is designed to take a fixed proportion of the flow over the main weir or spillway. This is normally achieved by sitting the invert level of the uppermost notch lower than the adjoining weir or spillway crest; the discharge and head calculations may be achieved by using appropriate weir – notch or orifice formulae.

The entrance to the fish pass (ladder) must be located in the downstream parallel to the main flow, whereas the exit (at the upstream end) should be well within the reservoir away from the spillway structure. The pools are usually 1 – 2 m deep, 2 – 5 m long and 2 – 10 m wide, depending on the number of fish migrating. Cross-walls may be provided with staggered notches–orifices, with the velocities in the ladder being around 0.5m/s. (Guiny, 2005) investigated the efficiency of different passages through the baffles in a fish pass (orifice, slot, weir) and found – with limited data – a strong preference of the migrating juvenile Atlantic salmon for the orifice type.



(a) Sectional elevation



(b) Plan

Figure 5.2: Fish ladder

5. Canal Head Regulator (Intake)

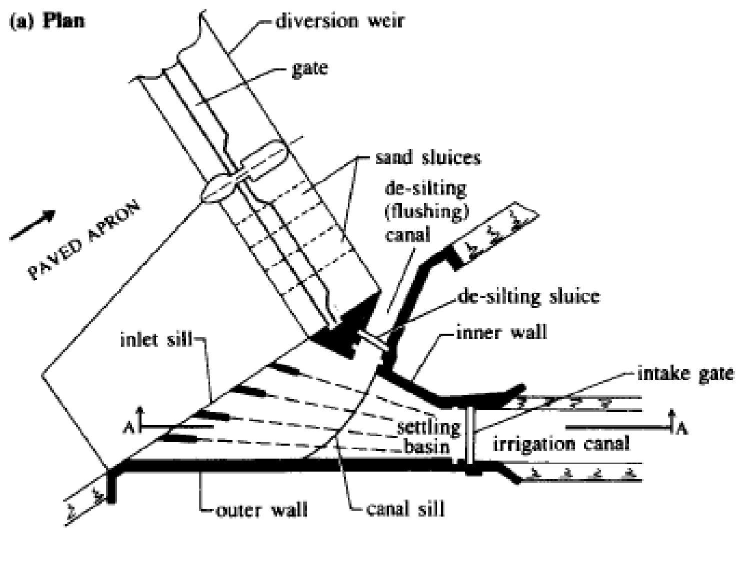
The intake structure (or head regulator) is a hydraulic device constructed at the head of an irrigation or power canal, or a tunnel conduit through which the flow is diverted from the original source such as a reservoir or a river. In other word the structures controlling diversion into a supply canal are called regulators. The design principles are the same as those used in the design of barrages, except that the regulators are a smaller version of barrages. The entry sill of a regulator must be such that it permits entry of the maximum flow at various poundage levels. Another important consideration in designing the regulator is silt exclusion from canals. Silt-excluder tunnels are often provided in the barrage bays adjacent to the regulator, so that the heavier silt-laden bottom layers of water bypass through the tunnels Figure. 5.3. The main purposes of the intake structure are:

- To admit and regulate water from the source, and possibly to meter the flow rate,
- To minimize the silting of the canal, i.e. to control the sediment entry into the canal at its intake,
- To prevent the clogging of the entrance with floating debris.

In high-head structures the intake can be either an integral part of a dam or separate; for example, in the form of a tower with entry ports at various levels which may aid flow regulation when there is a wide range of fluctuations of reservoir water level. Such a provision of multilevel entry also permits the withdrawal of water of a desired quality.

The layout of a typical intake structure on a river carrying a heavy bed load is shown in Figure. 5.3. The following are its major appurtenances:

- The raised inlet sill to prevent entry of the bed load of the river.
- The skimmer wall (with splitter pier) at the inlet to trap floating ice and debris
- The coarse rack (trash rack) to trap subsurface trash, equipped with either manual or automatic power-driven rack cleaning devices
- The settling basin (sand trap) followed by a secondary sill (entrance sill) diverting the bottom (sediment-laden) layers towards the de-silting canal
- The flushing (de-silting) sluice to flush the deposited silt
- The intake (head regulator) gates to control the flow rate into the canal
- The scouring tunnel (silt excluder) or undersluices in the diversion weir to flush the bed load deposited in upstream of the inlet sill.



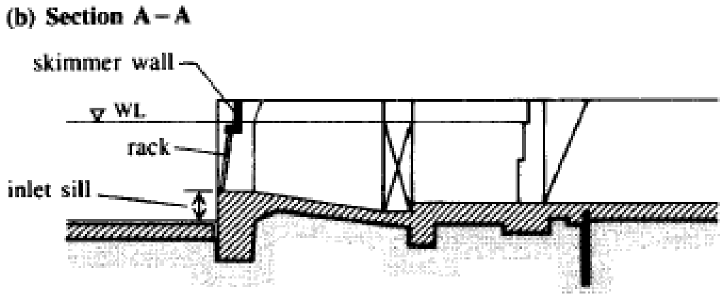


Figure 5.3: Canal intake on a river carrying heavy bed load (after Mosoni, 1987)

6. Approach Canal

The portion of canal from intake to settling basin is approach canal. Which drawn water from intake. This canal should withstand the negative pressure created by water because the velocity of water at this section is high. This canal should be stable against higher velocity for this purpose this canal is design by using either Manning's relation or Chezy's relation.

7. Silt Excluders (See section 5.5.4)

8. Silt Ejector (See section 5.5.5)

9. River Training Works (Marginal Bunds and Guide Bunds)

River training works direct the main river flow as centrally as possible to the diversion structure. They also safeguard the barrage from erosion and may be designed so that a desirable curvature is induced to the flow for silt exclusion from the canals. The side slopes of the guide banks must be protected by stone pitching, with a sufficient 'self-launching' stone apron at the lowest feasible level (Detail in chapter 6). The top levels of the guide banks will depend on the maximum increase in the flood level upstream of the barrage. The afflux (level difference between the headwater and tail water during the passage of maximum flood flow) results in a backwater curve upstream of the barrage, and flood banks have to be provided along the upstream reach of the river to contain the flood flow.

10. Stilling Basin (Energy Dissipater)

Stilling basins may be defined as the structure in which the energy dissipating action is confined. If the phenomenon of hydraulic jump is basically used for dissipating this energy, it may be called a hydraulic jump type stilling basin. The auxiliary devices like chute blocks, baffle piers, sill and dented sills may be used as additional measures for controlling the jump. For the particular site, the type of energy dissipater and its arrangement shall be decided based on the relationship between the height of hydraulic jump vs the tail water depth.

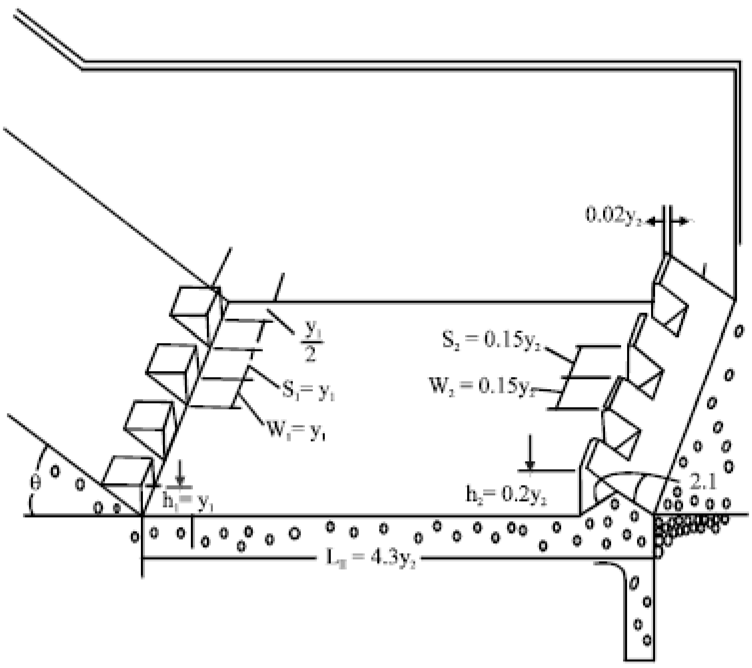


Figure 5.4: USBR stilling basin if (Fr_1>4.5)

5.3 Bligh's, Lane's and Khosla's Seepage Theory

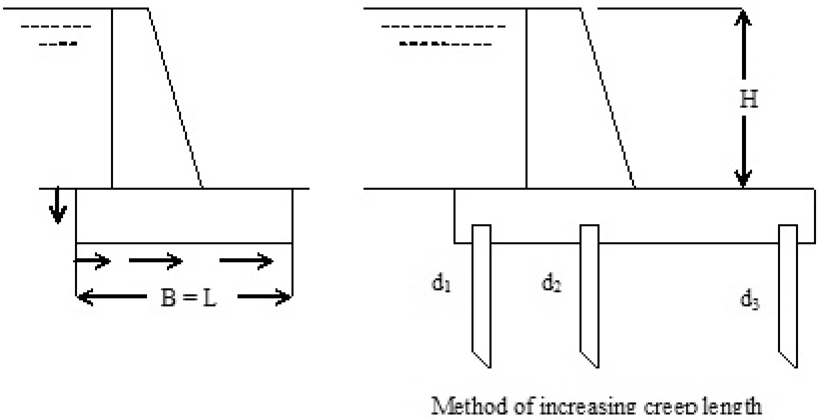
5.3.1 Bligh's Theory

In 1910, W.G. Bligh went a step forward and gave creep theory. According to this theory, the percolating water creeps along the contact surface of the base profile of the structure with the subsoil. The length of the path thus traversed by the percolation water is called the length of creep or the creep length. As the water creeps from the upstream end to the downstream end, the head loss occurs. The head loss is proportional to the creep distance travelled.

Bligh assumed that the water which percolates into the foundation creeps through the joint between the profile of the base of weir and the subsoil. Of course water also percolates into the subsoil. He then stated that this percolating water loses its head en-route. The seeping water finally comes out at the downstream end. According to Bligh water travels along vertical, horizontal or inclined path without making any distinction.

The total length covered by the percolating water till it emerges out at the downstream end is called a creep length. It is clear from the knowledge of hydraulics that the head of water lost in the path of percolation is the difference of water levels on the upstream and the downstream ends. Also, an imaginary line which joins the water levels on the upstream and the downstream end is called a hydraulic gradient line. Figure 5.6 (a, b) gives the full explanation of Bligh's theory.

According to Bligh's theory, the total creep length for first drawing: $L = B$ and for second drawing: $L = B + 2(d_1 + d_2 + d_3)$



(a) Only concrete floor (b)Method of increasing creep length with sheet piles

Figure 5.5: Creep Length due to Subsurface Flow

If H is the total loss of head, then the loss of head per unit length of the creep shall be

$$C = \frac{H}{L} = \frac{H}{[B + 2\,(d_1 + d_2 + d_3)]} \qquad \ldots\ 5.1$$

Bligh called the loss of head per unit length of creep as Percolation coefficient. The reciprocal, (L/H) of the percolation coefficient is known as the coefficient of creep C or Bligh's coefficient C_B.

Assumptions:

- Hydraulic slope or gradient is constant throughout the impervious length of the apron.
- The percolating water creep along the contact of the base profile of the apron with the sub soil head loss in creep path, proportional to length of its travel. The length is called creep length. It is the sum of horizontal and vertical creep length.
- Stoppage of percolation by cut off or piles is possible only if it extends up to impermeable soil strata.

Safety against Piping

The length of creep should be sufficient to provide safe hydraulic gradient according to safe creep length = $L = C_B \times H$

Table 5.2: Values of Bligh's percolation gradient for different types of soils

Soil type	Value of C_B	Safe hydraulic gradient should be less than
Silt and sandy clay	18	1/18
Misaceous sand	15	1/15
Coarse grained sand	12	1/12
Sand mixed with boulder, Gravel with shingle and Loamy soil	5 to 9	1/5 to 1/9
Gravel	5	1/5

Safety against Uplift Pressure

The second criterion for the safe design of a hydraulic structure on a pervious foundation is that the weight of the impervious floor must counterbalance the uplift pressure. The floor is usually designed as a gravity section.

Figure 5.6 (a) shows a simple horizontal floor length L, subjected to a seepage head of H. the residual head (h) at any point P is given by

$$h = H - \frac{H}{L} \times l \qquad \ldots\ 5.2$$

Where l is the horizontal length from the entry point A to the point P.

The residual head (h) can also be obtained from the subsoil hydraulic gradient line. It may be noted that the vertical intercept h of the subsoil HGL is measured from the top surface of the floor. It h' the vertical intercept measured above the bottom surface of the floor.

$$h' = h + t$$

Where t is the thickness of the floor.

From Figure 5.6 (b) shows the uplift pressure diagram on the bottom surface.

It is more convenient to measure the intercept h than the intercept h'. The intercept h' above the bottom surface of the floor can be determined only after the thickness t has been determined or t has been assumed. For the determination of the floor thickness t. let us consider the force acting on the unit area of the floor (shown hatched).

The upward force U due to the uplift pressure is given by

$$U = \gamma_w h' = \gamma_w (h + t) \qquad \ldots\ 5.3$$

The downward force W due to the weight of the floor material is given by

$$W = (G\,\gamma_w) \times t \qquad \ldots\ 5.4$$

Where G is the specific gravity of the floor material.

For equilibrium, the upward force should be counterbalanced by the weight. Hence from equation 5.3 and 5.4.

$$U = W$$

$$\gamma_w h' = G\,\gamma_w\, t$$

As $h' = h + t,$ $\qquad \gamma_w (h + t) = G\,\gamma_w\, t$

$$t\,(G - 1) = h$$

$$t = \frac{h}{(G - 1)} \qquad \ldots\ 5.5$$

Generally a factor of safety of 4/3 is adopted. Thus $t = \dfrac{4\,h}{3\,(G - 1)}$ $\qquad \ldots\ 5.6$

Equation 5.6 gives required thickness t of the floor if the height h of the subsoil hydraulic gradient line above the top surface of the floor is known. In this equation, G is the specific gravity of the floor material. For plain concrete floor, the value of G usually varies from 2.0 to 2.30, depending upon the type of aggregate used. For the design of irrigation structure, a value of 2.24 is generally adopted.

For portion of floor upstream of barrier only nominal thickness need to be provided since the weight of water will counterbalance the uplift pressure. A certain minimum length of impervious floor is always necessary to the downstream of the barrier (thickness of downstream floor for worst condition).

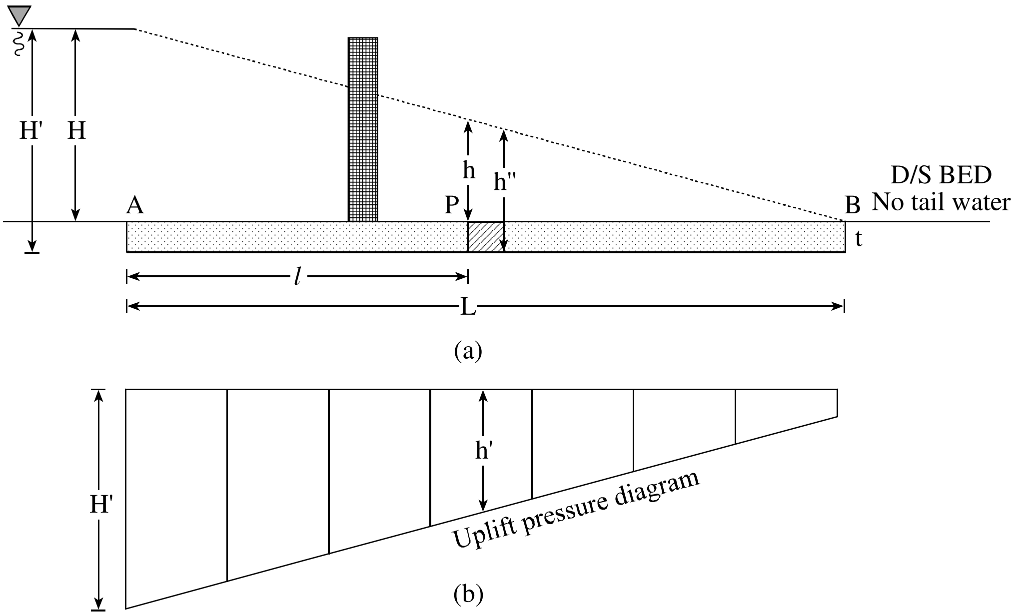


Figure 5.6: Bligh's theory

Limitations of Bligh's theory

Bligh's theory has several limitations. They are:

i. In his theory Bligh made no distinction between horizontal and vertical creep lengths.

ii. The idea of exit gradient has not been considered.

iii. The effect of varying lengths of sheet piles not considered.

iv. No distinction is made between inner or outer faces of the sheet piles.

v. Loss of head is considered proportional to the creep length which in actual is not so.

vi. The uplift pressure distribution is not linear as assumed but in fact it follows a sine curve.

vii. Necessity of providing end sheet pile not appreciated.

5.3.2 Lane's Theory

Lane analyzed a large number of dams and weir founded on pervious foundations which failed or did not fail. He brought out deficiencies in Bligh's creep theory and gave a new theory on statistical basis. The theory is known as Lane's weighted creep theory. This theory gives the weightage factor of 1/3 for the horizontal creep, as against 1.0 for the vertical creep.

From Figure 5.5 b. $L = B/3 + (2d_1+2d_2+2d_3)$

Safety against Piping

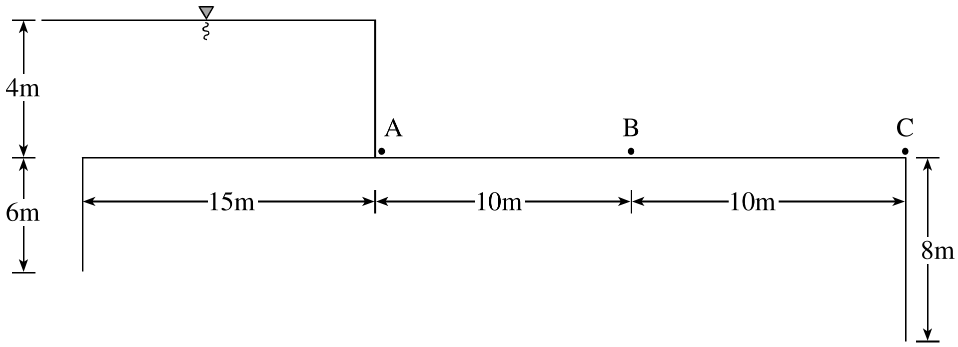
To ensure safety against piping, according to this theory, the creep length L, must not be less than $C_l H_L$, where H_L is the head causing flow, and C_L is Lane's creep coefficient given in table below.

Table 5.3: Values of Lane's safe Hydraulic Gradient for different types of soils.

S.N.	Types of soil	Value of Lane's Coefficient C_L	Safe Lane's Hydraulic gradient should be less than
1.	Very fine sand or silt	8.5	1/8.5
2.	Fine sand	7.0	1/7
3.	Coarse sand	5.0	1/5
4.	Gravel and sand	3.5 to 3.0	1/3.5 to 1/3
5.	Boulders, gravels and sand	2.5 to 3.0	1/2.5 to 1/3
6.	Clayey soils	3.0 to 1.6	1/3 to 1/1.6

Example 5.1:

Figure shows a hydraulic structure built on fine sand. Determine, whether the percolation gradient is safe? Also determine uplift pressure and floor thickness using G = 2.24 at point A, B and C. Assume Bligh's C_B = 15 and Lane's C_L = 8.5.



Solution:

(a) From Bligh's theory

i. Creep length L= 2 × 6 + 35 + 2 × 8=63m.

Hydraulic gradient $i = H/L = 4/63 = \frac{1}{15.75} < \frac{1}{15}$ safe

ii. Uplift pressure head

At point A $= 4 - \frac{1}{15.75}\ (2 \times 6 + 15) = 2.29\text{m}$

At point B $= 4 - \frac{1}{15.75}\ (2 \times 6 + 25) = 1.65\text{m}$

At point C $= 4 - \frac{1}{15.75}\ (2 \times 6 + 35) = 1.02\text{m}$

Check uplift pressure head at point C from end point $= \frac{1}{15.75} \times 2 \times 8 = 1.02\text{m}$ OK

iii. Thickness of floor

We have, $t = \dfrac{4\,h}{3\,(G-1)}$

At point A, $t = \dfrac{4 \times 2.29}{3\,(2.24-1)} = 2.46$ m

At point B, $t = \dfrac{4 \times 1.65}{3\,(2.24-1)} = 1.77$ m

At point C, $t = \dfrac{4 \times 1.02}{3\,(2.24-1)} = 1.10$ m

(b) From Lane's theory

Weighted creep length, $L_w = 35/3 + 2 \times 6 + 2 \times 8 = 39.67$m

Hydraulic gradient = $H/L_w = 4/39.67 = 1/9.92 < 1/8.5$ (safe)

Uplift pressure at point A = $4 - (2 \times 6 + 15/3)/9.92 = 2.29$m

At point B = $4 - (2 \times 6 + 25/3)/9.92 = 1.95$m

At point C = $4 - (2 \times 6 + 35/3) = 1.61$ m

[Check uplift pressure at point C = $(2 \times 8)/9.92 = 1.61$m Ok]

Thickness of floor At point A = $\dfrac{4}{3}\ \dfrac{2.29}{2.24-1} = 2.46$m

At point B = $\dfrac{4}{3}\ \dfrac{1.95}{2.24-1} = 2.10$m

At point C= $\dfrac{4}{3}\ \dfrac{1.61}{2.24-1} = 1.73$m

It may be noted that the uplift pressure are quite different from those obtained by Bligh's theory basically in down stream and Lane's thickness is in safe side.

5.3.3 Khosla's Theory

The seeping water does not creep along the bottom contour as stated by Bligh and Lane, it moves along a set of streamlines as shown in Figure 5.7.

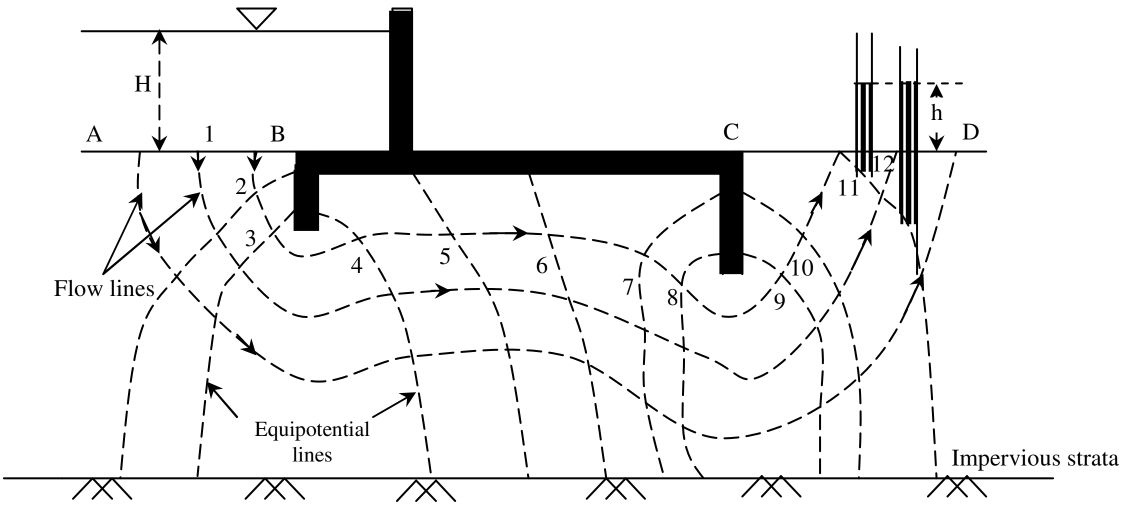


Figure 5.7: Khosla's flow net

This steady seepage in a vertical plane for a homogeneous soil can be expressed by Laplacian equation:

$$\frac{d^2\phi}{dx^2} + \frac{d^2\phi}{dz^2} = 0$$

Where, ϕ – flow potential. ϕ = kh; where, k is the coefficient of permeability as defined by Darcy and h is the residual head at any print within the soil.

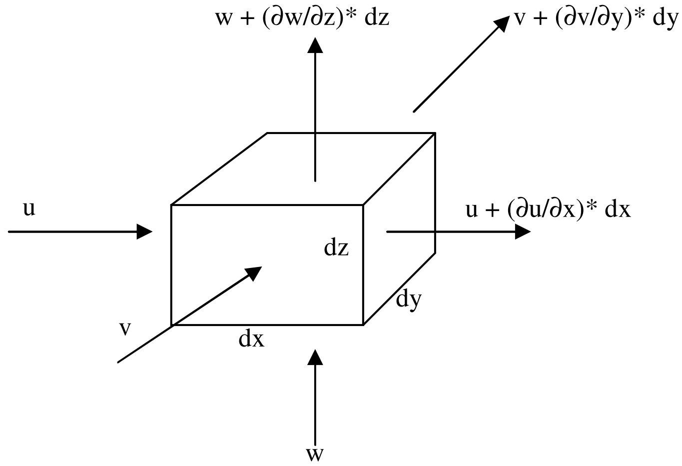


Figure 5.8: Expression of Laplace equation

Quantity of water entering per unit time = u dz dy + v dx dz + w dx dy . . . . (i)

Quantity leaving per unit time:

$$\left(u + \frac{\partial u}{\partial x} dx\right) dzdy + \left(v + \frac{\partial v}{\partial y} dy\right) dzdx + \left(w + \frac{\partial w}{\partial z} dz\right) dxdy \qquad \text{. . . . (ii)}$$

If the flow is steady & incompressible, the quantity entering is equal to quantity leaving.

$$\left(\frac{\partial u}{\partial x} + \frac{\partial v}{\partial y} + \frac{\partial w}{\partial z}\right) dxdydz = 0$$

Since, dx dy dz $\neq$ 0;

$$\frac{\partial u}{\partial x} + \frac{\partial v}{\partial y} + \frac{\partial w}{\partial z} = 0 \qquad \text{. . . . (iii)}$$

Now, Darcy law is given by

$$\mathrm{V} = \mathrm{K}\frac{\mathrm{dh}}{\mathrm{dx}}$$

Hence, $u = k\dfrac{\partial h}{\partial x}, v = \dfrac{\partial h}{\partial y}, \& w = \dfrac{\partial h}{\partial z}$

Further, ϕ = kh, so that

$$u = \frac{\partial \phi}{\partial x}, v = \frac{\partial \phi}{\partial y}, \& w = \frac{\partial \phi}{\partial z},$$

Putting their value in equation (iii)

$$\frac{\partial^2 \phi}{\partial x^2} + \frac{\partial^2 \phi}{\partial y^2} + \frac{\partial^2 \phi}{\partial z^2} = 0 \qquad \text{. . . . (iv)}$$

Equation (iv) is known as Laplace’s equation

For two-dimensional flows:

$$\frac{d^2 \phi}{dx^2} + \frac{d^2 \phi}{dz^2} = 0$$

Above equation represents 2 sets of curves intersecting each other orthogonally (see Figure: 5.7). AB is the first equipotential line and CD is the last.

The residual head causes drag or seepage pressure at any point. The determination of residual pressure (uplift) is possible by flowing methods:

- Trial and error or graphical method
- Mathematical solution of Laplace's equation
- Khosla’s method of independent variables
- Method of electrical analogy
- Method of relaxation

Why Khosla’s method is preferable than other methods?

- Difficult to plot the flow net graphically
- All other methods are complicated & time consuming
- Khosla's method is simple, quick & accurate

The usual barrage and weir section do not conform to a simple elementary form and a direct solution of the Laplace equation not feasible. To apply the analytical solution to any practical composite profile of a weir or a barrage, Khosla’s and his associates evolved the method of independent variables. In this method a composite barrage or weir section is split up into a number of simple standard forms of known analytical solutions. The most useful standard forms among these are.

a. A straight horizontal floor of negligible thickness with a sheet pile at either end (Figure 5.9 (a) and (b))

b. A straight horizontal floor depressed below the bed but no vertical cutoff. (Figure 5.9 (c))

c. A straight horizontal floor of negligible thickness with a sheet pile line at some intermediate position. (Figure 5.9 (d))

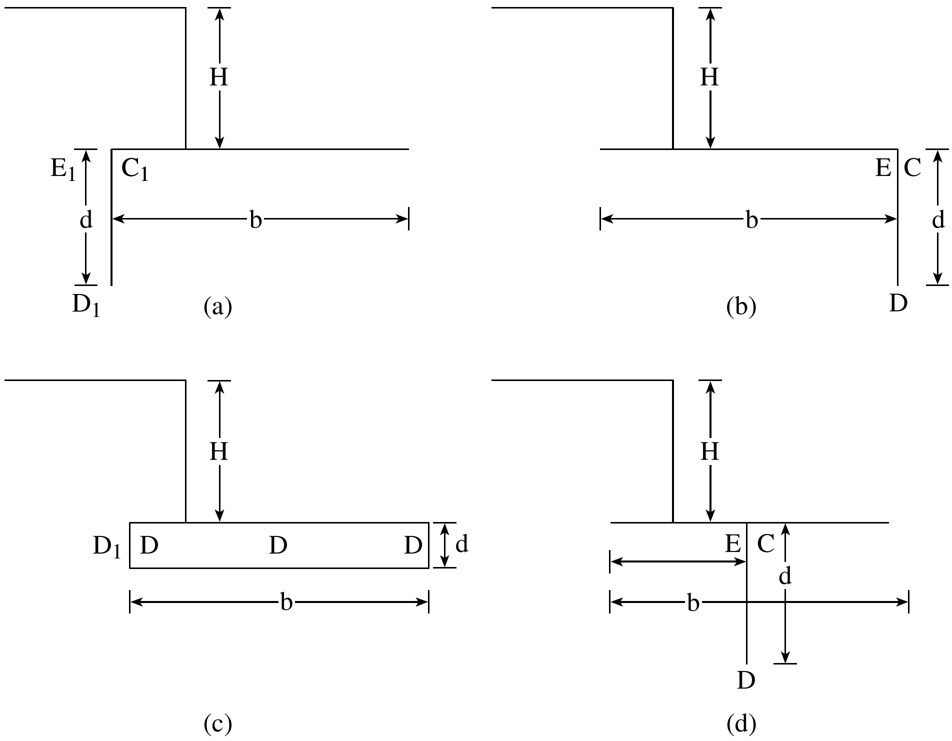


Figure 5.9: Standard forms – Khosla's methods of independent variables

In general, the usual weir section consists of a combination of all the three forms mentioned above; the entire length of the floor with any of the pile lines, etc. making up one such form. Each elementary form is then treated as independent of the others. The pressures at the key points are then read off from the Khosla's curves (Annex I). These key points are the junction points of the floor and the pile line of that particular elementary form, the bottom point of that pile line and the bottom corners in the case of depressed floor.

The percentage pressure observed from the curves for the simple form into which the profile has been broken up, is valid for the profile as a whole if corrected for:

(i) Mutual interference of sheet piles

(ii) The floor thickness

(iii) The slope of the floor

(i) Correction for mutual interference of piles

Let C, be the correction to be applied as percentage of head, b' be the distance between the two pile lines. D be the depth of the pile line, the influence of which has to be determined on neighboring pile of depth d. D is to be measured below the level at which interference is desired. d be the depth of pile on which the effect is to be determined.

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

This correction is positive for points in the rear of back water and subtractive for points forward in the direction of flow.

In the case of intermediate piling shallower than the end pile (D <d) and b' $\geq$ 2d the mutual interference is negligible.

(ii) Correction for floor thickness

In the standard forms with vertical cut off the thickness of the floor is assumed to be negligible. Thus as observed from the curves, the pressure at the junction point E and C pertain to the level at the top of the floor whereas the actual junction is with the bottom of the floor. The pressures at the actual points E and C are interpolated by assuming straight line variation from hypothetical point E to D and also from D to C.

(iii) Correction for the slope of the floor

A suitable percentage correction is to be applied for a sloping floor, the correction being plus for the down and minus for the up slopes following the direction of flow. The values of the correction are given in Table 5.4

Table 5.4: Correction for the floor slope

Slope V:H	Correction % of Pressure
1:1	11.2
1:2	6.5
1:3	4.5
1:4	3.3
1:5	2.8
1:6	2.5
1:7	2.3
1:8	1

The correction is applicable to the key points of the pile line fixed at the beginning or the end of the slope. The percentage correction given by the above table is to be further multiplied by the proportion of the horizontal length of slope to the distance between the two pile lines in between which the sloping floor is located. The correction is minus for the up and plus for the down slopes in the directions of flow.

Exit Gradient (G_E)

It has been determined that for a standard form consisting of floor of length b, with a vertical cutoff of depth, d, the exit gradient at its downstream end is given by the equation;

$$G_E = \frac{H}{d}\frac{1}{\pi\sqrt{\lambda}} \qquad \ldots 5.7$$

Where, $\lambda = \dfrac{1 + \sqrt{1 + \alpha^2}}{2}$ . . . 5.8

$\alpha = \dfrac{b}{d}$

From the curves in Annex II for any value of α or b/d, the corresponding value of $\dfrac{1}{\pi\sqrt{\lambda}}$ can be read off.

Knowing the values of H and d, the value of G_E is easily calculated. It is obvious from equation 5.5 that if d = 0, G_E is infinite. It is therefore, essential that a vertical cutoff should be provided at the downstream end of the floor. To safeguard against piping, the exist gradient of soils are given in Table 5.5. The uplift pressures must be kept as low as possible consistent with the safety at the exit, so as to keep the floor thickness to the minimum.

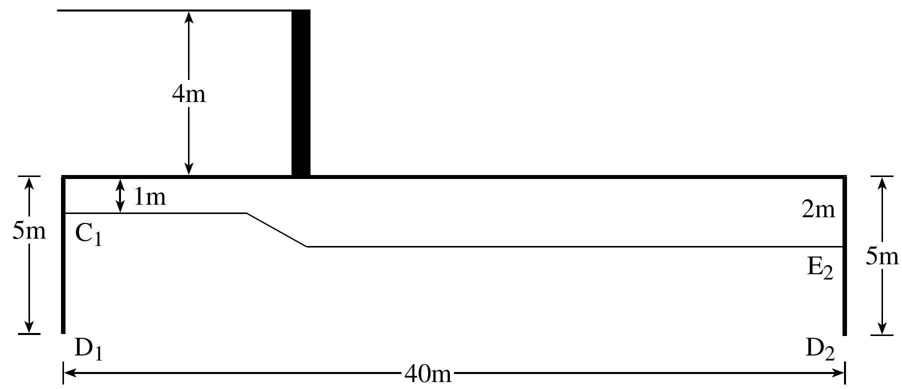
Table 5.5: Permissible exit gradients for different types of soils

Types of soil	Exit gradient
Find sand	1/6 to 1/7
Coarse sand	1/5 to 1/6
Shingle	¼ to 1/5

Example 5.2:

A weir has a solid horizontal floor length of 40m with two lines of cutoff 5m depth below the river bed at its two ends. The floor thickness is 5m at the upstream end and 2m at downstream end, with its upper level being in flush with the river bed. For an effective head of 4m over the weir, calculate the uplift pressure at the two inside corner points (junction of bottom of floor with the cutoff) and also exit gradient.

Solution:



Given: Maximum barrage head (Hs) = 4m

Length of weir (b) = 40m

U/S and D/S cutoff depth (d, and d_2) = 5m

For D/S cutoff:

$$\alpha = \frac{b}{d} = \frac{40}{5} = 9$$

$$\lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2} = \frac{1 + \sqrt{1 + 8^2}}{2} = 4.531$$

$$\phi_{E_2} = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right) = \frac{1}{\pi}\cos^{-1}\left(\frac{4.531 - 3}{4.531}\right) = 0.3113 = 31.3\%$$

$$\phi_{D_2} = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda - 1}{\lambda}\right) = \frac{1}{\pi}\cos^{-1}\left(\frac{4.531 - 1}{4.531}\right) = 0.2155 = 21.55\%$$

Correction due to thickness $= \dfrac{\phi_E - \phi_D}{d} \times t = \dfrac{31.13 - 21.55}{5} \times 2 = 3.83\%$

Correction due to interference of U/S cutoff, given by,

$$C = -19\sqrt{\frac{D}{b'}}\left(\frac{d + D}{b}\right) = -19\sqrt{\frac{5}{40}}\left(\frac{5 + 5}{40}\right) = -1.68\%$$

Hence, correct $\phi_E = 31.13 - 3.83 - 1.68 = 25.62\%$

Uplift pressure at $E_2 = 0.2562 \times 4 = 1.02$ %

For C/S cutoff,

Since, $d_1 = d_2$, α and λ are same as D/S cutoff

Hence, $\phi_{C_1} = \phi_{E_2} = 100 - 31.13 = 68.87\%$

$\phi_{D_1} = 100 - \phi_{D_2} = 100 - 21.55 = 78.45\%$

Correction for thickness, $= \dfrac{\phi_{D_1} - \phi_{C_1}}{d} \times t = \dfrac{78.45 - 68.87}{5} \times 1 = 1.926\%$

Correction due to interference of D/S cut of given by

$$C = 19\sqrt{\frac{D}{b'}}\left(\frac{d + D}{b}\right) = 1.68\%$$

Hence, corrected $\phi_{C_1} = 68.87 + 1.92 + 1.68 = 72.47\%$

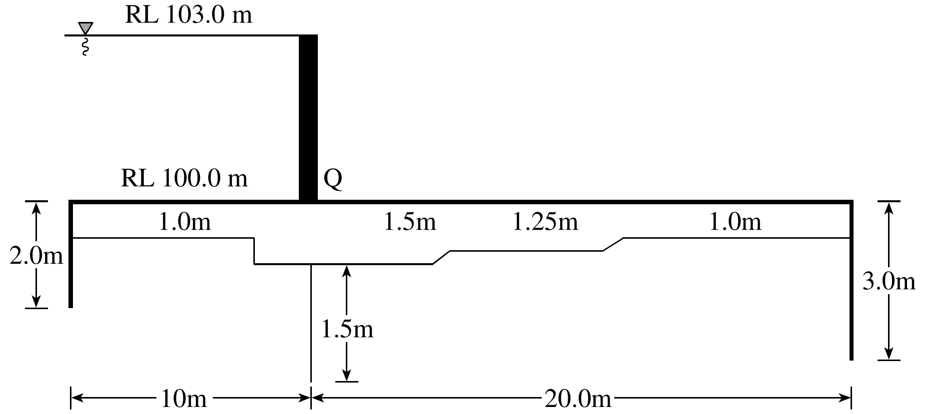
Uplift pressure at $C_1 = 0.7247 \times 5 = 3.62$ m

Exit radiant,

$$G_E = \frac{H.S.}{d}\frac{1}{\pi\sqrt{\lambda}} = \frac{5}{8}\frac{1}{\pi\sqrt{4.531}} = \frac{1}{10.7}$$

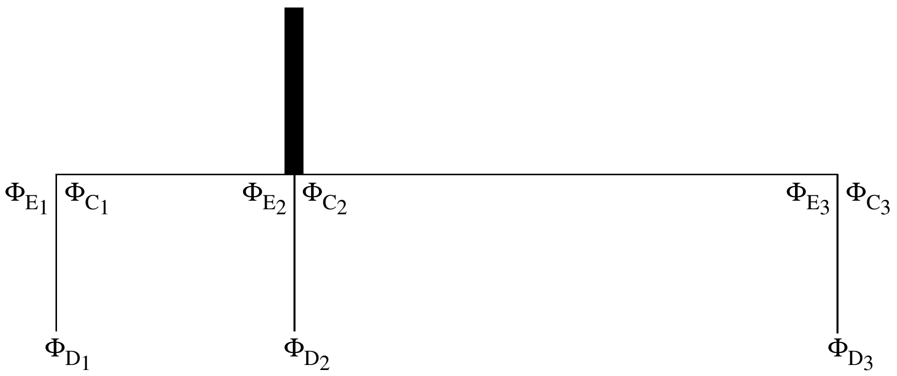
Example 5.3:

Calculate the uplift pressure at key points of the pile of the structure shown below. Also check the thickness provided and safe exit gradient $G_E = 1/5$.



Solution:

Let split composite profile into simple profile,



Pile 1 Pile 2 Pile 3

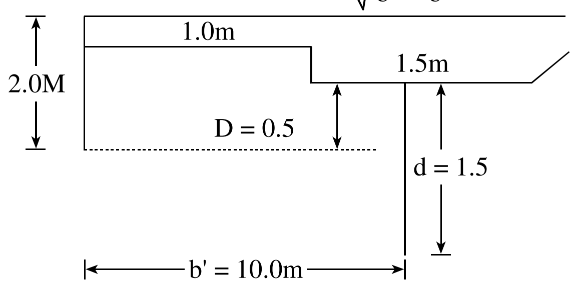
For pile no 1	For pile no 2	For pile no 3
b= 30m d=2m $\alpha = b/d = 15$ $\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = 8.02$ $\Phi_{E1} = \frac{100}{\pi}\cos^{-1}\frac{0-\lambda}{\lambda} = 100\%$ $\Phi_{D1} = \frac{100}{\pi}\cos^{-1}\frac{1-\lambda}{\lambda} = 83.93\%$ $\Phi_{C1} = \frac{100}{\pi}\cos^{-1}\frac{2-\lambda}{\lambda} = 77.02\%$	b_1= 10m b_2=20m d=3m $\alpha_1 = b_1/d = 3.33$ $\alpha_2 = b_2/d = 6.67$ $\lambda_1 = \frac{\sqrt{1+\alpha_1^2} - \sqrt{1+\alpha_2 2}}{2} = -1.634$ $\lambda = \frac{\sqrt{1+\alpha_1^2} + \sqrt{1+\alpha_2 2}}{2} = 5.11$ $\Phi_{E2} = \frac{100}{\pi}\cos^{-1}\frac{\lambda_1-1}{\lambda} = 67.24\%$ $\Phi_{D2} = \frac{100}{\pi}\cos^{-1}\frac{\lambda_1 - 0}{\lambda} = 60.36\%$ $\Phi_{C2} = \frac{100}{\pi}\cos^{-1}\frac{\lambda_1+1}{\lambda} = 53.96\%$	b= 30m d=3m $\alpha = b/d = 10$ $\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = 5.52$ $\Phi_{E3} = \frac{100}{\pi}\cos^{-1}\frac{\lambda - 2}{\lambda} = 27.99\%$ $\Phi_{D3} = \frac{100}{\pi}\cos^{-1}\frac{\lambda - 1}{\lambda} = 19.46\%$ $\Phi_{C3} = \frac{100}{\pi}\cos^{-1}\frac{\lambda - 0}{\lambda} = 0\%$

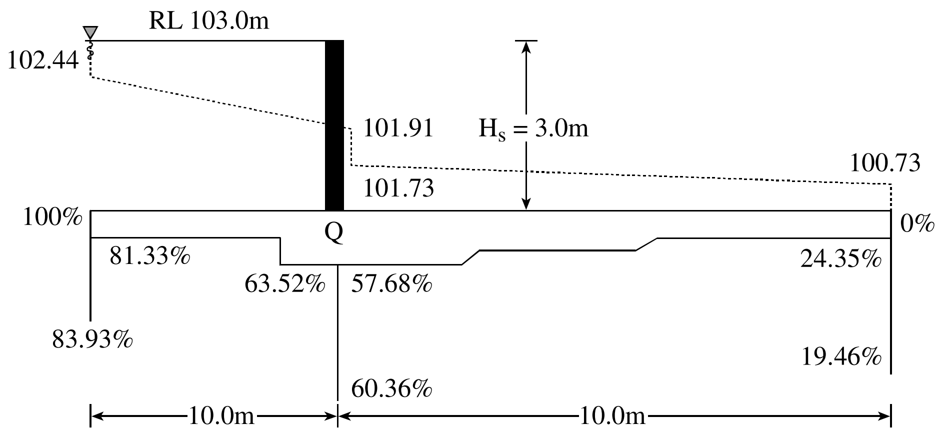
Point	Pressure Φ% (1)	Correction for Thickness (2)% $=\frac{\Phi D-\Phi C}{d}\times t$ or $\frac{\Phi D-\Phi E}{d}\times t$	Correction for Mutual interference (3) % $=\pm 19\sqrt{\frac{D}{b'}}\frac{d+D}{b}$	Correction for Slope % (4)	Corrected Pressure Φ% (5) = (1) + (2) + (3) + (4)	Pressure Head (m) (6) = (5) × H_s/100	HGL (masl) = 103 - Hs+ (6)
E_1	100	0	0	0	100	3	103
D_1	83.93	0	0	0	83.93	2.52	102.52
C_1	77.02	3.46	0.85	0	81.33	2.44	102.44
E_2	67.24	-3.44	-0.28	0	63.52	1.91	101.91
D_2	60.36	0	0	0	60.36	1.81	101.81
C_2	53.96	3.20	0.52	0	57.68	1.73	101.73
E_3	27.99	-2.84	-0.80	0	24.35	0.73	100.73
D_3	19.46	0	0	0	19.46	0.58	100.58
C_3	0	0	0	0	0	0	100.0

Sample calculation

Correction for thickness at point $E_2 = \frac{\Phi D-\Phi E}{d}\times t = \frac{60.36-67.24}{3}\times 1.5 = -3.44$

Correction for mutual interference at point $E_2 = -19\sqrt{\frac{D}{b'}}\frac{d+D}{b} = -19\sqrt{\frac{0.5}{10}}\frac{1.5+0.5}{30} = -0.28$





Thickness required

Pressure at point Q at the beginning of D/S floor = 1.91m [Take maximum pressure head at that point]

Thickness required $= \frac{h}{G-1} = \frac{1.91}{2.24-1} = 1.56\text{m}$

Provided thickness = 1.50m < 1.56 hence unsafe.

Thickness at the end of floor $= \frac{0.73}{2.24-1} = 0.588$

Provided thickness = 1m > 0.588. Hence safe.

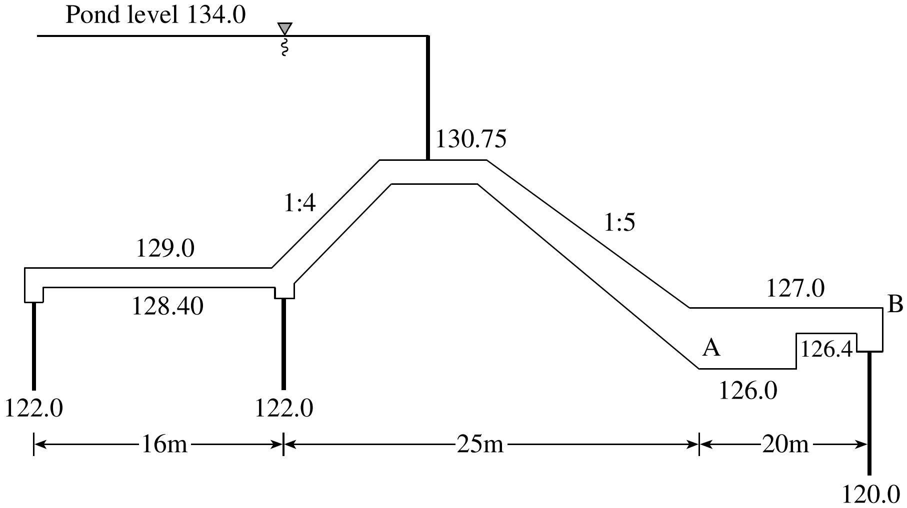
Exit Gradient

$G_E = \frac{H}{d}\frac{1}{\pi\sqrt{\lambda}}$ H = 3m

d = 3m $\lambda = 5.52$ $G_E = 1/7.38 < 1/5$ (safe)

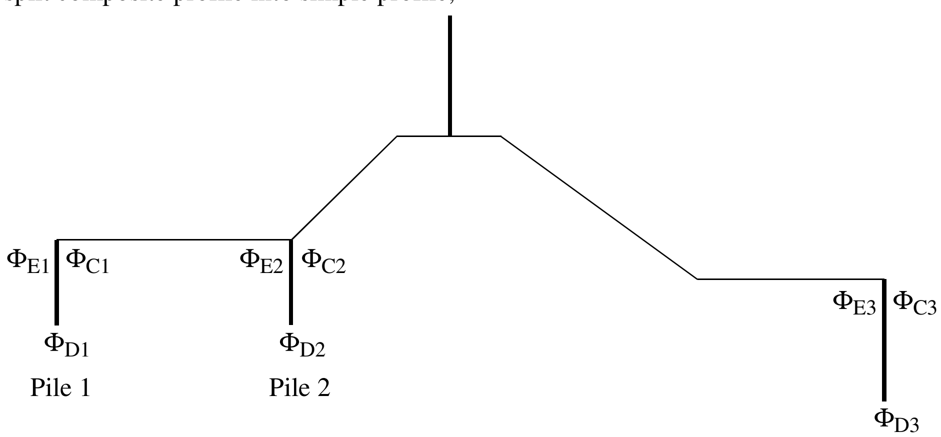
Example 5.4:

Find whether the section is safe against uplift at A, also determine the safe against piping.



Solution:

Let split composite profile into simple profile,



For pile no 1	For pile no 2	For pile no 3
b= 61m d=129 – 122 = 7m α = b/d= 8.714 $\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = 4.885$ $\Phi_{E1} = \frac{100}{\pi}\cos^{-1}\frac{0-\lambda}{\lambda} = 100\%$ $\Phi_{D1} = \frac{100}{\pi}\cos^{-1}\frac{1-\lambda}{\lambda} = 79.26\%$ $\Phi_{C1} = \frac{100}{\pi}\cos^{-1}\frac{2-\lambda}{\lambda} = 70.11\%$	b_1 = 16m b_2 =45m d =129 – 122 = 7m $\alpha_1 = b_1/d$ $\alpha_2 = b_2/d$ $\lambda_1 = \frac{\sqrt{1+\alpha_1^2} - \sqrt{1+\alpha_2 2}}{2} = -2.0$ $\lambda = \frac{\sqrt{1+\alpha_1^2} + \sqrt{1+\alpha_2 2}}{2} = 4.5$ $\Phi_{E2} = \frac{100}{\pi}\cos^{-1}\frac{\lambda_1-1}{\lambda} = 73.28\%$ $\Phi_{D2} = \frac{100}{\pi}\cos^{-1}\frac{\lambda_1 - 0}{\lambda} = 64.70\%$ $\Phi_{C2} = \frac{100}{\pi}\cos^{-1}\frac{\lambda_1+1}{\lambda} = 57.171\%$	b = 61m d =127 – 120 = 7m α = b/d= 8.714 $\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = 4.885$ $\Phi_{E3} = \frac{100}{\pi}\cos^{-1}\frac{\lambda - 2}{\lambda} = 29.88\%$ $\Phi_{D3} = \frac{100}{\pi}\cos^{-1}\frac{\lambda -1}{\lambda} = 20.73\%$ $\Phi_{C3} = \frac{100}{\pi}\cos^{-1}\frac{\lambda - 0}{\lambda} = 0\%$

Point	Pressure Φ% (1)	Correction for			Corrected Pressure Φ% (5) = (1) + (2) + (3) + (4)	Pressure Head (m) (6) = (5) × H_s/100	HGL (masl) = 134 – Hs + (6)
		Thickness (2)% $= \frac{\Phi D - \Phi C}{d} \times t$ or $\frac{\Phi D - \Phi E}{d} \times t$	Mutual interference (3) % $= \pm 19\sqrt{\frac{D}{b'}}\frac{d+D}{b}$	Slope % (4)			
E_1	100	0	0	0	100	7	134.00
D_1	79.26	0	0	0	79.26	5.54	132.54
C_1	70.11	0.78	2.52	0	73.41	5.13	132.13
E_2	73.28	– 0.73	– 2.52	0	70.02	4.90	131.90
D_2	64.70	0	0	0	64.70	4.53	131.53
C_2	57.17	0.64	1.99	– 0.51	59.29	4.15	131.15
E_3	29.88	– 0.78	– 1.05	0	28.04	1.96	128.96
D_3	20.73	0	0	0	20.73	1.45	128.45
C_3	0	0	0	0	0	0	127.00

Sample calculation

Correction for thickness at point $C_2 = \frac{64.70 - 57.17}{7} \times 0.6 = 0.645\%$

Correction for mutual interference

Effect of pile no 3 on pile no 2

D =128.4 – 120 = 8.4m

d= 128.4 – 122.0 = 6.4m

b'= 45m

b = 61m

$$\text{Correction} = 19\sqrt{\frac{D}{b'}}\frac{d+D}{b} = 19\sqrt{\frac{8.4}{45}}\frac{6.4+8.4}{61} = 1.99\%$$

Correction for slope

Slope correction at point $C_2 = C_s \times \frac{b_s}{b'}$

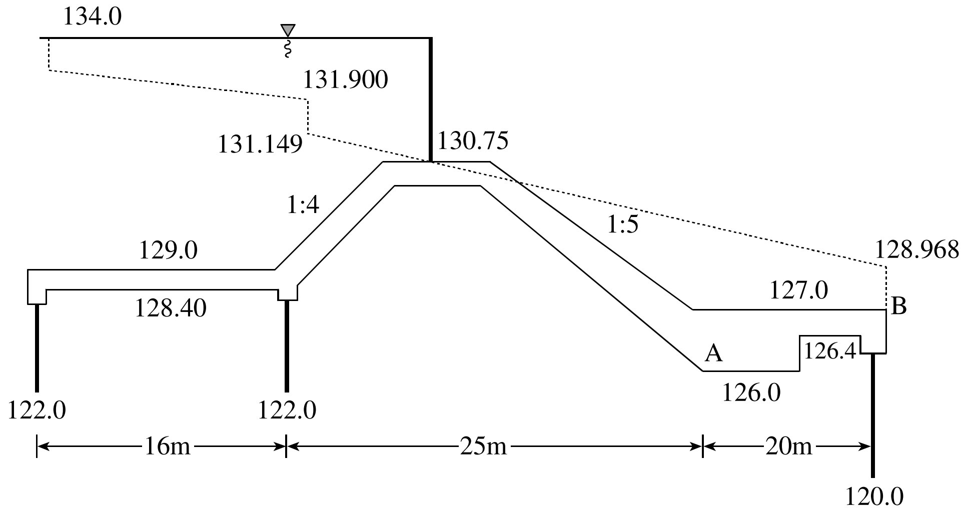
Vertical height = 130.75 – 129.0 = 1.75m

Sloping length b_s= 1.75 × 4 = 7m

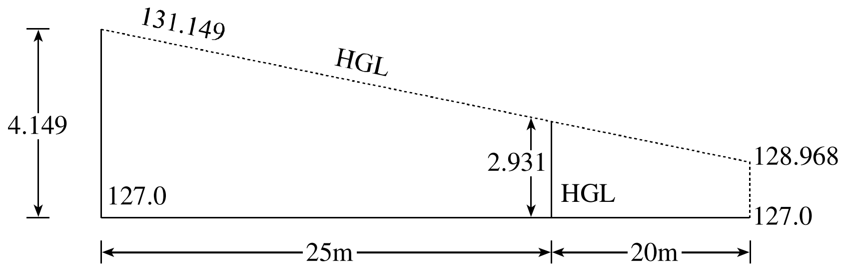
b' = 45m

C_s = 3.3 for 4:1 slope (from Table 5.4)

Correction = $C_s \times \frac{b_s}{b'}$ = – 0.513% (negative for upward)



Check for uplift press at A



Now, uplift pressure at A, h = 2.931m

Thickness required at A, $t = \frac{h}{G - 1} = \frac{2.931}{2.24 - 1}$ = 2.363. But provided thickness is = 1m Hence unsafe.

Now check for exit gradient

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

H = 7 m

d = 127 – 120 = 7 m

λ = 4.18

$G_E = \frac{7}{7} \frac{1}{\pi \sqrt{4.18}}$ = 1/6.42 < 1/5. Hence, the weir is safe against piping.

5.4 Design of Sloping Glacis Weir Bay

Stepwise procedure for the design of the Barrage

Data needed:

i. Maximum flood discharge Q
ii. Stage discharge curve of the river at barrage site
iii. Minimum water level
iv. Cross – section of the river at barrage site.

The following have to be decided:

- Lacey's silt factor $f = 1.76\sqrt{d}$ where d = mean diameter of particles in mm.
- Length of waterway, discharge per meter and afflux.
- Safe exit gradient
- Depth of sheet piles in relation to
  - Scour depth
  - Exit gradient
- Level and layout of horizontal part of downstream impervious floor in coordination with hydraulic jump.
- Thickness of downstream impervious floor:
  - With reference to uplift pressures.
  - With reference to hydraulic jump or standing wave.
- Length and thickness of protection works beyond PCC floor upstream and downstream.

Procedure:

Step 1: Determine head loss H_L for different flow conditions.

If there is no retrogression, H_L = afflux.

If allowance for retrogression is taken in downstream bed level, then H_L = Afflux + retrogression. Usually 0.5m retrogression will be sufficient in most cases.

Step 2: For known values of q and H_L, read corresponding values of Ef_2 from Blench curves (Annex III), with known values of Ef_2 read corresponding values of d_2.

Cistern level = downstream T.E.L. – Ef_2

Step 3: $Ef_1 = Ef_2 + H_L$, knowing Ef_1, Ef_2 and q read values of d_1 and d_2 from Annex IV energy of flow curves. Provide minimum cistern length $5(d_2 – d_1)$; desirable $6(d_2 – d_1)$.

Step 4: Determine scour depths from the formula:

$$R = 1.35(q^2/f)^{1/3} \qquad \ldots 5.9$$

Depth of upstream sheet pile from scour consideration = (1 to 1.25) × R

Depth of downstream sheet pile from scour consideration = (1.25 to 1.5) × R.

An intermediate sheet pile line need not normally be provided. If at all provided, its depth should not be less than that of the upstream pile line.

Step 5: Work out the value of $\frac{1}{\pi\sqrt{\lambda}}$ from the equation $\frac{1}{\pi\sqrt{\lambda}} = G_E\frac{d}{H}$ for the given value of G_E and the known value of d (downstream sheet pile) and H (maximum static head). Corresponding to the value of $\frac{1}{\pi\sqrt{\lambda}}$, read the value of d from Annex II

Step 6: Provide total length of floor $b = \alpha d$.

Deposition of total floor length may be as follows:

– Cistern length = $5(d_2 - d_1)$ to $6(d_2 - d_1)$

– Glacis length = 3 to 5 times (crest level – cistern level) from 3:1 to 5:1 slope of glacis plus crest width.

– Upstream floor = the balance.

If the total length is excessive, it would be economical to reduce it by providing a deeper downstream sheet pile.

Step 7: In order to determine uplift pressure acting on the floor, the % pressures at upstream and downstream sheet pile lines are worked out. The pressure distribution from upstream sheet pile line to downstream sheet pile line is assumed to be linear.

Correction due to floor thickness

The thickness of the floor at the location of the sheet piles are tentatively assumed for correcting the values of Φ_C in the upstream and Φ_E in the downstream. If t_1 is the floor thickness at upstream sheet pile of depth d_1, correction due to floor thickness = $t_1/d_1\ (\Phi_D - \Phi_C)$ which is positive. If t_2 is the floor thickness at downstream sheet pile of depth d_2, the correction = $t_2/d_2\ (\Phi_E - \Phi_D)$ which is negative.

Correction due to mutual interference of sheet piles

The correction due to mutual interference of sheet pile is worked out by the formula

$$C = \pm 19\sqrt{\frac{D}{b_1}}\left(\frac{d + D}{b}\right)$$

Correction due to slope

This is applicable only in case where an intermediate pile line is provided. The values of correction are discussed earlier.

Step 8: After knowing the corrected percentage pressures under the key points the sub – soil pressure gradient line and hydraulic gradient line for surface flow is plotted with reference to the corresponding downstream water level as datum. The corresponding water profiles before and after the jump formation are also plotted for the given value of discharge intensity q.

Knowing q and Ef_1 at different location of the glacis, corresponding values of d_1 are read from annex III and thus the water profile before jump formation can be plotted. For plotting water profile after jump, the Froud number Fr is determined from the relation

$$Fr = \frac{q}{\sqrt{g d_1^{\,2}}}$$

Knowing Fr relation between the abscissa and ordinate of the profile can be read from the Annex IV.

The uplift pressure which will occur with the maximum pond level upstream and no flow downstream should also be determined. The requirement of floor thickness is worked out by taking the larger of the two uplift pressure and dividing it by (G – 1), G being the density of floor material and (G – 1) the submerged density of floor material.

Step 9: The protection works are designed for the scour depth will discussed in chapter 6.

5.5 Sediment Control Measures and Devices at the Headwork

The rivers flowing in erodible valleys carry heavy sediment load during floods. The channel which take off from these rivers draw heavy sediment which they cannot carry due to their slopes being milder than that of the river. This results in silting of the channel in head regulators. The entry of coarse particles in a power channel may in addition cause damage to the turbine blades.

In a flowing river the sediment concentration in the bottom portion is much higher and coarser than that in the upper layers. If the bottom layers are removed without disturbing the natural sediment distribution in the stream large quantity of sediment in suspension, saltation or moving on the bed shall be removed. The aim of all sediment control and exclusion measures and devices is to prevent or remove sediment concentrated in the bottom layers so that the channels are left with sediment near to their sediment-carrying capacity. In power channels particles larger than 0.25mm is generally required to be removed to prevent damage to the turbine blades (Varshney, 2001).

5.5.1 Classification of sediment control measures and devices

The various means of controlling sediment into the canal and ejecting that enter the canal may be categorized as follows:

1. **Sediment preventive measures**

   Entry of sediment into the channel can be discouraged by adopting certain preventive measures which are generally incorporated in the layout of headworks itself. These include:

   (i) River approach conditions.
   (ii) Alignment of head regulators
   (iii) Alignment of guide bunds
   (iv) Length of divide wall
   (v) Relative crest levels of under sluices and the head regulator and
   (vi) River regulation

2. **Sediment control devices**

   The bed load moves in the lower layer of flow. A number of devices in the form of submerged bars have been evolved to diver the bed load away from the head regulator. Some devices have been conceived with the idea of deflecting upper layers of flow from the main channel into the canal.

   The devices may be classified as under:

   - Guide vanes
   - Skimming platform and
   - Vortex tube

3. **Sediment exclusion devices**

   The following devices are included in this category:

   (i) Sediment excluder
   (ii) Sediment ejector
   (iii) Settling basin

   Sediment excluders are constructed in the river pocket adjacent to the head regulator in order to exclude the coarse sediment by removing bottom layers of river water. Sediment ejector is constructed in the head of the canal to eject the slit which may enter the canal inspite of sediment control measures taken and sediment exclusion devices provided. Settling basins are found to be quite effective in case of low dams which work on the principle that at a low velocity the sediment is deposited in the basin and clear water is drawn into the canal or water conductor system.

5.5.2 Sediment Preventive Measures

I. River approach condition

The curvature of the river flow approaching the canal head regulator is very important from the point of view of sediment entry into the canal. It has been observed that the canal head regulator located near the downstream end of a concave curvature is the best for minimum sediment entry. The river channel leading to the head regulator should however not be very deep.

II. Alignment of head regulator

It has been observed that generally setting back of the upstream abutment of the head regulator from the line at right angles to the barrage axis is helpful in reducing sediment entry into the canal.

In past, head regulators have been generally aligned at 100 to 105 degrees with barrage axis. However recent model studies have indicated that the head regulators at an angle between 105 to 110 degrees with the barrage axis are optimum for minimum sediment entry into the canal.

III. Alignment of guide bunds

Suitable alignment of guide bunds helps in generating favorable curvature of flow for minimum sediment entry into canal. It has been observed that the influence of guide bunds in reducing the sediment is considerably reduced due to formation of islands upstream of the barrage which is a common feature in wider barrages. The exact alignment depends on the river approach condition.

IV. Length of divide wall

A divide wall at the end of under sluices helps in creation of favorable flow conditions. In old headworks very long divide walls have been provided. However, it has been observed that very long divide walls are not advantageous. Divide wall extending up to 2/3rd the width of the head regulator in case of one head regulator, and extending up to the end of second head regulator is considered optimum.

V. Relative crest levels of undersluices and head regulator

Crest level of undersluices are kept lower than other barrage bay about 1 m. this helps in maintaining well defined channel in front of head regulator, the crest level of head regulator is kept 1 m to 4 m higher than the crest of undersluices. Raised crest of the regulator is found effective in reducing sediment entry into the canal in case the canal discharge is a small proportion of the river discharge.

VI. River regulation

The river regulation involves the sequence of gate operation for passing the flood discharge through various barrage bays. The river regulation has a bearing on the sediment entry into the canal. The usual practices of river regulation are:

(a) Still-pond – when the undersluices bays are entirely closed and flood is passed through other barrage bays. The canal thus draws the discharge from the still pond in the pocket.

(b) Semi-still pond regulation- when the gates in the undersluices pocket are partially opened and canal is drawing the discharge from a flowing stream.

(c) The open-flow regulation- when all the gates in the undersluices pocket are fully open. This condition is conductive to more sediment entry in canal. Both the model and prototype observations have proved the superiority of still-pond regulation over other methods.

If still pond regulation in which frequent flushing of the pocket is required cannot be adopted especially on power channels, semi-still pond regulation with progressive increased opening of gates away from the head regulator may be adopted.

5.5.3 Sediment Control Devices

Guide vanes in the form of submerged bars may divert the bed load away from the head regulator. This arrangement can be used for distributary heads. Descriptions of various types of vanes are as follows:

I. King's Vanes

These silt vanes are vertical diaphragm walls parallel to each other, starting in line with the current and terminating at an angle with it. They are at low height, so as to diver bottom layers of water loaded with sediment away from the head regulator.

II. Skimming Platform

It consists of a slab supported on piers so as to exclude silt laden bottom water and leaves such quantity of top water that should fill the offtake. The best shape of the platform is as shown in figure. However one or both ends can be made square.

The following criteria of design may be adopted.

(i) The minimum depth should be 0.6 m deeper tunnels are preferred.

(ii) Width of platform should be sufficient so that enough water passes over it to fill the offtake with some spare say 20 to 25%.

(iii) Pier number may be decided from structural considerations. The pier noses should slope 1 in 3. The noses should be cut water shape.

(iv) The downstream edge of the platform may be made at an angle of 60° to the center line of the parent channel. The upstream edge may be parallel to the other edge but it should be at 1.5 to 3.0 m "upstream of the upstream" edge at the off-take.

III. Vortex Tube

It consists of an open tube placed across the bottom of the canal either normal to it or at an angle greater than 30°. To control the flow out of the tube, its downstream end is regulated by a valve. The upper portion of the tube is removed to trap the sediment. As water passes over the tube, a shearing action across the open portion sets up a vortex motion within the tube, this has sufficient velocity to prevent sediment deposition in tube.

5.5.4 Sediment Excluders

The first approach to control sediment entry into the canals was made by F.V. Elsden in 1922. He suggested diversion of the bottom layers of flow through the tunnels back into the river by putting a diaphragm at a suitable height without disturbing the sediment distribution. However, the idea took a practical shape in 1934 when H. W.Nicholson constructed an excluder in the pocket of Lower Chennab canal at Khanki head works. The design consisted of six tunnels of different lengths covering the full length of the regulator and discharging into first two layers of the undersluices.

5.5.5 Sediment Ejectors or Extractors

Despite measures which may be taken to prevent sediment entry in the canal, a part of the suspended sediment of the river would always enter the canal. If the sediment transporting capacity of the canal is not adequate to carry this sediment, the canal would be silted. In such cases ejectors are provided in the head reaches to eject part of the coarse sediment that entered the canal.

The the principle of design of various components of the ejector are the same as for excluder.

5.5.6 Settling basins or sedimentation chamber

Provision of silt excluder is possible in barrages having their crests close to the river bed. In case of high dams most of the sediment particles settle down in the reservoir and no exclusion devices are required. In case of low dams which have very little trap efficiency, it is generally necessary to construct sediment exclusion devices. In such dams, silt excluders cannot be provided and only silt excluders are possible. When it is desired to exclude very fine suspended particles, settling basins or tanks are used for sediment exclusion. The low lying area along the canal is converted into a basin by constructing bunds. Silt water enters from one side and clear water flows back into the canal through the other end. The sediment which drops down in the basin is either mechanically removed or is flushed out.

Settling tanks or sedimentation chambers have been provided on many irrigation and multipurpose projects in the country to remove every particle. The recent examples being Seti multipurpose project Kaski, Trisuli hydropower project and etc.

5.5.7 Advantages and Disadvantages of Excluders and Ejectors

According to F.F Haigh (1934) the advantages and disadvantages of excluders and ejectors are as given below:

Advantages of Excluders:

(i) The head across the barrage for its operation is usually available.

(ii) Economy is affected by the use of barrage gates and cisterns,

(iii) Large orifices unlikely to be checked by rolling submerged floating debris can be provided easily.

Disadvantages of Excluders:

(i) The difficulty of securing good approach conditions.

(ii) The undersluices bays covered by the excluders cannot be used simultaneously for the passage of flood discharge.

(iii) The structure being subject to the river action has to be robust.

Advantages of Ejectors:

(i) Good approach conditions are secured with ease.

(ii) The extent of extraction can be improved by constructing a series of ejectors in the canal.

Disadvantages of Ejectors:

(i) There is difficulty in securing a working head with High River and low canal discharges.

(ii) The size of the head regulator has to be increased to pass additional discharge for the escape.

(iii) The canal section has to be increased upstream of the ejector to accommodate the escape discharge.

(iv) Comparatively small orifices are liable to be blocked by debris and may necessitate the provision of trash rack at the canal head regulator.

Despite various advantages and disadvantages of excluders and ejectors, it may be essential to provide both devices in important canal system.

5.6 Design of Under Sluice and Silt Excluder

5.6.1 Under sluices

Discharge over weir – bay and sluice bay sections:

Before starting the actual design of weir, the discharge passing over the weir – bay section and that over the undersluice section should be decided.

The discharge over the undersluice section should be equal to or greater than the greatest of the following:

- It should be at least equal to twice the full supply discharge of the offtaking canal.
- It should be greater than the dry weather flow of the ricer during winter so that the weir crest gates (or shutters) are not required to be opened.
- It should be a substantial portion of the total design discharge so that small floods can be passed over the sluice section, without lifting the weir crest gates. It is the usual practice to take the discharge over the sluice section about 20% of the total discharge and the remaining 80% discharge over the weir – bay section.

Design Procedure

The design procedure is similar for the undersluice section and the weir bay section. However, the levels of the two portions are different. Generally, Khosla's theory is used for design. The procedure may be summarized as follows:

(i) Fix the discharge over the weir – bay and undersluice bay sections, as discussed above.

(ii) Fix the crest level of the undersluice section and the weir bay section.

- The crest of the undersluice section is usually kept flush with upstream floor. The slope of the D/S glacis is usually kept between 3:1 and 5:1.
- In the case of weir bay section, as the crest level is higher than the bed level, an upstream glacis is also provided with a slope of 1:1 to 3:1. The top width of the crest is kept about 2.0m to 3.0m.

(iii) Fix the water way for the weir – bay and undersluice sections.

(iv) Determine the characteristics of the hydraulic jump for the high flood condition and the pond level condition, with:

- No flow concentration and no retrogression and
- Blow concentration and with retrogression.

(v) Calculate the normal scour depth and determine the bottom levels of the upstream and downstream piles. If necessary, provide an intermediate pile.

(vi) Find the total length of the impervious floor from the exit gradient consideration. Also fix the lengths and levels of the upstream and downstream floors. The D/S floor level is fixed below the point at which the hydraulic jump is formed.

(vii) Calculate the percentage uplift pressure at the key points of all piles by Khosla's theory for the following conditions.

- No flow condition
- High flood condition and pond level condition, without flow concentration and without retrogression.
- High flood condition and pond level condition, with flow concentration and with retrogression. Draw the subsoil hydraulic gradient line (H.G.L.) for each case.

(viii) Determine the uplift pressure at various points from the subsoil H.G.L. for low flow condition. Also determine the suction pressure at the jump through for high flood condition and pond level condition.

(ix) Calculate the thickness of the floor at various points for the maximum of the uplift obtained for the three conditions mentioned in step vii.

(x) Provide concrete block protection on the upstream and downstream sides.

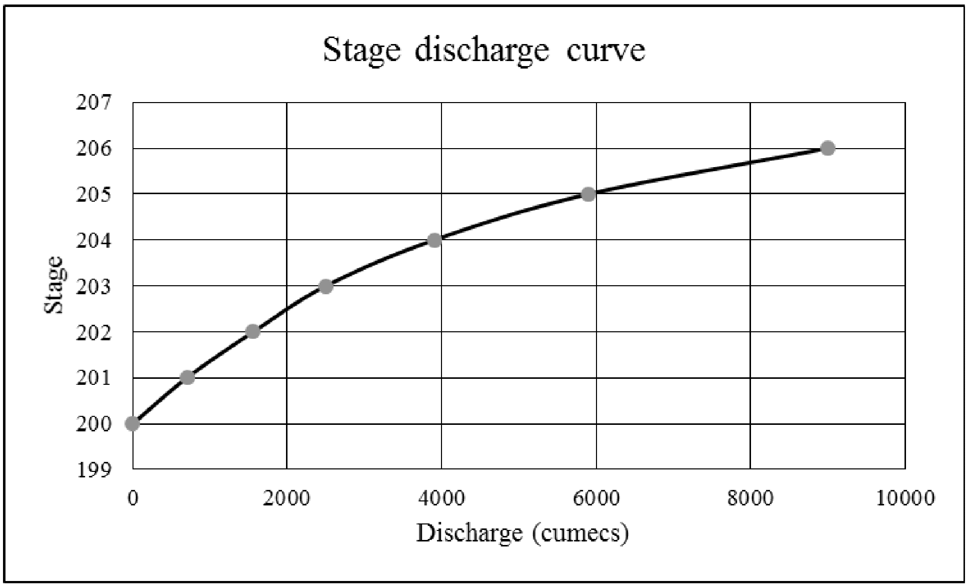
(xi) Provide launching aprons on the upstream and downstream sides.

Example 5.5:

Design the undersluice section of a diversion headworks with the following data:

- Design flood discharge in the river = 9000 m^3/sec
- Deepest bed level of the river = 200.00
- High flood level before construction = 206.00
- Full supply level of canal = 203.00
- Permissible afflux = 1.0m
- Bed retrogression = 0.50m
- Discharge concentration factor = 20%
- Lecey's silt factor = 1.0
- Safe exit gradient = 1/6
- Pier contraction coefficient = 0.10
- Full supply discharge of canal = 200.0 m^3/sec

The stage discharge curve of the river at the site is shown below. Assume suitable data if necessary.



Solution:

Step 1: Crest level

Crest level of undersluice = deepest bed level = 200.00

Crest level of weir bay = 200.00 + 1.5 = 201.50

Pond level = F.S.L. + modular head = 203.00 + 1.0 = 204.00

Step 2: Water way

From Lacey's formula, $P = 4.75\sqrt{Q} = 4.75\sqrt{9000} = 450.62m$

Let us fix the water way as follows:

(a) Undersluice portion

Number of spans = 5

5 bays of 16 m each = 80m

4 pier of 2.5 m each = 10m

Total = 90m

(b) Weir bay portion

Number of spans = 25

25 bays of 12 m each= 300m

24 pier of 2 m each = 48m

Total = 348 m

Width of fish ladder = 5m

Width of divide wall = 3m

Overall waterway between abutments = 90 + 348 + 5 + 3 = 446m

This waterway is approximately equal to P.

Average discharge intensity q= 9000/446 = 20.18 cumecs/m

Step 3: Discharge

Let us first determine the U/S T.E.L.

U/S HFL after construction = H.F.L. before construction + afflux = 206 + 1.0 = 207.0m

Normal scour depth from Lacey's formula = $R = 1.35\left(\frac{q^2}{f}\right)^{\frac{1}{3}} = 1.35\left(\frac{20.18^2}{1.0}\right)^{\frac{1}{3}} = 10.00m$

Velocity of approach = q/R = 20.18/10 = 2.02 m/s

Head due to velocity of approach= $\left(\frac{v^2}{2g}\right) = 0.21m$

U/S T.E.L. = U/S HFL + Head due to velocity of approach = 207.00 + 0.21 = 207.21

(a) Undersluices

Head over the undersluices crest = 207.21 – 200 = 7.21

Since the U/S floor and the crest of the undersluices are at the same level, the width of crest is large and it will behave as a broad crested weir. Therefore, discharge is given by:

$Q = 1.705\ (L' - 0.1 \times n\ H_e)\ H_e^{3/2}$

Where n is the number of end contraction. Assuming that the end near the divide wall and the abutment are suppressed, n = 8

$Q = 1.705\ (80 - 0.1 \times 8 \times 7.21)\ 7.21^{3/2} = 2450.30$ cumec

(b) Weir – bay

Head over the weir bay crest = 207.21 – 201.50 = 5.71m

Let us assume the crest width of weir as 2m. As the crest width $B < \frac{2H}{3}$, the weir will act as a sharp crested weir. Therefore discharge is given by

$Q = 1.84\ (L' - 0.1 \times n \times H_e)\ H_e^{3/2} = 1.84\ (300 - 0.1 \times 48 \times 5.71)\ 5.71^{3/2} = 6843.61$ cumec

Total discharge Q = 2450.30 + 6843.61 = 9293.91 cumecs > 9000 cumecs **OK**

Let us verify whether the discharge through undersluice is adequate.

Twice the design full supply discharge of canal = 400 cumecs

20% of the total design flood discharge = 1800 cumecs

The discharge capacity of undersluice is more than the minimum required by above two criteria and may be adopted. As the dry weather flow is not given the third criterion cannot be applied.

Step 4: Design of undersluice section

Let us find out the discharge intensity q and head loss H_L for different conditions.

(a) High flood conditions

(i) Without flow concentration and without retrogression

$q = 1.705\ (H_e)^{3/2} = 1.705 \times (7.21)^{3/2} = 33.00$ cumecs/m

D/S TEL = 206.00 + 0.21 = 206.21

U/S TEL = 207.00 + 0.21 = 207.21

Head loss, H_L = 207.21 – 206.21= 1.00m

(ii) With 20% flow concentration and retrogression of 0.5m

$q = 1.20 \times 33.0 = 39.6$ cumecs/m

H_e = Head over the crest = $(39.6/1.705)^{2/3} = 8.14$m

U/S TEL = 200.00 + 8.14 = 208.14

D/S TEL after retrogression = 206.21 – 0.5 = 205.71

Head loss, H_L= 208.14 – 205.71 = 2.43m

(b) Pond level conditions

(i) Without flow concentration and without retrogression

Head over the crest of undersluices = 204.0 – 200.0 = 4.0m

At the pond level condition, the velocity of approach can be found from the discharge which occurs at that level.

$$Q = 1.705\ (80 - 0.1 \times 8 \times 4.0)\ (4.0)^{3/2} + 1.84\ (300 - 0.1 \times 48 \times 2.5)\ (2.5)^{3/2}$$

or, $Q = 1047.55 + 2094.69 = 3142.24$ cumecs.

Average discharge intensity = 3142.24/446.00 = 7.05 cumecs/m

Normal scour depth, $R = 1.35\left(\frac{q^2}{f}\right)^{\frac{1}{3}} = 1.35\left(\frac{7.05^2}{1}\right)^{\frac{1}{3}} = 4.96\text{m}$

Velocity of approach, $v_a = q/R = 7.05/4.96 = 1.42$m/sec

Head due to velocity of approach = $\left(\frac{v_a^2}{2g}\right) = 0.10$m

U/S TEL = 204.0 + 0.1 = 204.10

Discharge intensity $q = 1.705 \times (4.10)^{3/2} = 14.15$ cumecs/m

The water level on the D/S of the weir when a discharge of 3142.24 cumecs occurs in the river is obtained from the stage discharge curve as 203.50m.

D/S TEL = 203.50 + 0.1 = 203.60

Head loss, $H_L = 204.10 - 203.60 = 0.50$m

(ii) With 20% flow condition and 0.5m retrogression

Discharge intensity = 1.2 × 14.15 = 16.98 cumecs/m

Head over the crest = $(16.98/1.705)^{2/3} = 4.63$m

U/S TEL = 200.00 + 4.63 = 204.63

D/S water level after retrogression = 203.50 – 0.5 = 203.00

D/S TEL = 203.00 + 0.1 = 203.10m

Head loss, $H_L = 204.63 - 203.10 = 1.53$ m

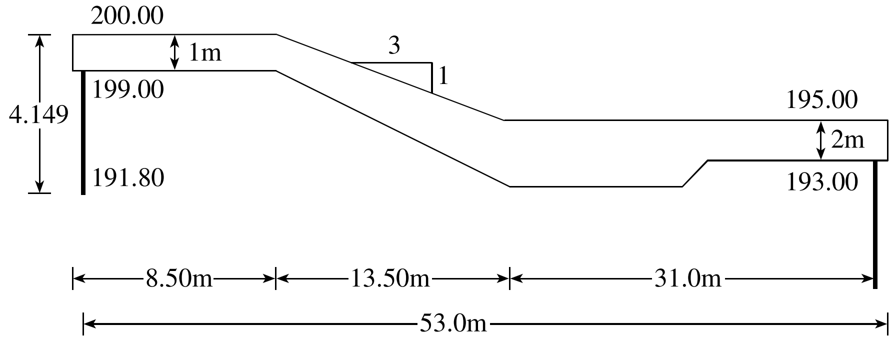
Step 5: Hydraulic jump calculation

The hydraulic jump calculation are given in table 5.6.

Table 5.6: Hydraulic jump calculation with different conditions

S.N.	Item	High flood condition		Pond level condition	
		No concentration and retrogression	with concentration and retrogression	No concentration and retrogression	with concentration and retrogression
1	Discharge intensity q cumecs/m	33.00	39.60	14.15	16.98
2	U/S water level	207.00	207.00	204.00	204.00
3	U/S TEL	207.21	208.14	204.10	204.63
4	D/S water level	206.00	205.50	203.50	203.00
5	D/S TEL	206.21	205.71	203.60	203.10
6	Head Loss (H_L)	1.00	2.43	0.50	1.53
7	Ef_2 from blench curve	8.60	10.10	4.60	5.80
8	Level at which the hydraulic jump is formed (5) - (7)	197.61	***195.61***	199.00	197.30
9	$Ef_1 = Ef_2 + H_L$ = (7) + (6)	9.60	12.53	5.10	7.33
10	Pre jump depth y_1(from montague's curve)	2.80	2.90	1.80	1.60
11	Post jump depth , y_2 (from montague curve)	7.60	9.00	3.80	5.30
12	length of horizontal floor $= 5(y_2 - y_1)$	24.00	***30.50***	10.00	18.50
13	Initial Froud no $= \frac{q^2}{\sqrt{gy^3}}$	2.25	2.56	1.87	2.68
14	Fr^2	5.06	6.55	3.50	7.10
15	$y_c = \left(\frac{q^2}{g}\right)^{\frac{1}{3}}$	4.80	5.43	2.73	3.09
16	$z = \frac{H_L}{y_c}$	0.21	0.45	0.18	0.50

The lowest water level at which the hydraulic jump is formed corresponds to the high flood condition with flow concentration and retrogression and is 195.61. Let us adopt the downstream floor level is 195.50. The greatest length of the horizontal floor is 30.50 m. Let us adopt the length of the D/S horizontal floor as 31.0m.



Step 6: Depth of sheet piles

Total discharge over undersluices = 2450.30 m^3/s

Overall waterway = 80 + 2.5 × 4 = 90m

Average discharge intensity = 2450.30/90 = 27.23 cumecs/m

Normal scour depth, $R = 1.35 \left(\frac{27.23^2}{1}\right)^{\frac{1}{3}} = 12.22m$

Let us take the maximum scour depth at D/S as 1.50R = 18.33m

Bottom level of D/S pile = D/S water level after retrogression – 1.5R = 205.50 – 18.33 = 187.17, say 187.00.

Depth of D/S pile below floor level, D_2 = 195.50 – 187.00 = 8.50m

Let us take the maximum scour depth at U/S as 1.25R = 15.20m

Bottom level of U/S pile = U/S water level – 1.25R = 207.00 – 15.20 = 191.80.

Depth of U/S pile below the river bed D_1 = 200.00 – 191.80 = 8.20m

Step 7: Length of impervious floor

Maximum seepage head H = Pond level – D/S floor level = 204.00 – 195.50 = 8.50m

We have, the safe exit gradient $G_E = \frac{H}{d}\frac{1}{\pi\sqrt{\lambda}}$ or $\frac{1}{6} = \frac{8.50}{8.50}\frac{1}{\pi\sqrt{\lambda}}$ or $\lambda = 3.648$

Now $\lambda = \frac{1+\sqrt{1+\alpha^2}}{2}$ or $3.648 = \frac{1+\sqrt{1+\alpha^2}}{2}$ or $\alpha = 6.22$

$b = \alpha\, d$ or $b = 6.22 \times 8.50 = 52.9m$

Let us adopt the overall length of 53 m as shown above figure.

Length of U/S floor = 8.50m

Length of D/S glacis = 13.50m

Length of D/S floor = 31.00m

Step 8: Uplift pressure calculations

Let us assume the thickness of U/S floor as 1.0m and that of the D/S floor at the end as 2.00m for the purpose of calculating the uplift pressures. Khosla's theory is used for uplift pressure calculations.

For pile no 1	For pile no 2
b= 53m d=8.20m $\alpha = b/d = 6.46$ $\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = 3.77$ $\Phi_{E1} = \frac{100}{\pi}\cos^{-1}\frac{0-\lambda}{\lambda} = 100\%$ $\Phi_{D1} = \frac{100}{\pi}\cos^{-1}\frac{1-\lambda}{\lambda} = 76.27\%$ $\Phi_{C1} = \frac{100}{\pi}\cos^{-1}\frac{2-\lambda}{\lambda} = 65.56\%$ i. Correction for thickness for $\Phi_{C1} = \frac{76.21-65.56}{8.20} \times 1.00 = 1.30$ ii. Correction for mutual interference $=19\left(\frac{d+D}{b}\right)\sqrt{\frac{D}{b'}} =$ $19\left(\frac{7.2+12}{53}\right)\sqrt{\frac{12}{53}} = 3.28\%$ Corrected Φ_{C1} = 65.56 + 1.30 +3.28 = **70.14%**	b= 53m d=8.5m $\alpha = b/d = 6.23$ $\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = 3.65$ $\Phi_{E2} = \frac{100}{\pi}\cos^{-1}\frac{\lambda-2}{\lambda} = 35.07\%$ $\Phi_{D2} = \frac{100}{\pi}\cos^{-1}\frac{\lambda-1}{\lambda} = 24.13\%$ $\Phi_{C2} = \frac{100}{\pi}\cos^{-1}\frac{\lambda-0}{\lambda} = 0\%$ i. Correction for thickness for $\Phi_{E2} = \frac{35.07-24.13}{8.50} \times 2.00 = 2.57\%$ ii. Correction for mutual interference $=19\left(\frac{d+D}{b}\right)\sqrt{\frac{D}{b'}}$ $=19\left(\frac{6.50+1.70}{53}\right)\sqrt{\frac{1.70}{53}} = 0.53\%$ Corrected Φ_{E2} = 35.07 – 3.95 – 0.53 = **31.97%**

The level of subsoil hydraulic gradient line HGL can be calculated assuming linear variation between key points C_1 and E_2 for different flow condition. As shown in table below.

Table 5.7 Uplift pressure calculation

S. No.	Flow condition	U/S water level	D/S water level	Seepage Head (m)	Height of subsoil HGL above D/S water level and its elevation					
					U/S pile			D/S pile		
					$\Phi_{C1=100}$	$\Phi_{C1=76.27\%}$	$\Phi_{C1=70.14\%}$	$\Phi_{C1=30.59\%}$	$\Phi_{C1=18.29\%}$	$\Phi_{C1=0\%}$
1	No flow	204	195.5	8.45	8.5	6.48	5.96	2.6	1.55	0
				RL	204	201.98	201.46	198.1	197.05	195.5
2	High flood with flow concentration and retrogression	207	205.5	1.5	1.5	1.14	1.05	0.46	0.27	0
				RL	207	206.64	206.55	205.96	205.77	205.5
3	Flow at pond level with flow condition and retrogression	204	203	1	1	0.76	0.7	0.31	0.18	0
				RL	204	203.76	203.7	203.31	203.18	203

Step 9: Hydraulic jump profile

For high flood condition and flow at pond level, the suction pressure is computed from the hydraulic jump profile.

(a) Pre jump profile

(b) High flood flow with 20% concentration and 0.5m retrogression.

(c) Pond level flow with 20% concentration and 0.5m retrogression.

The hydraulic jump forms at RL for 195.61 in the first case and 197.30 in the second case, as already calculated in table 5.8.

Table 5.8: Pre jump profile calculation

Distance from the end of crest (1)	R.L. of glacis (2)	High flood condition			pond level condition		
		U/S TEL = 208.14 q=39.60cumecs/m			U/S TEL = 204.63, q = 16.98 cumecss/m		
		Ef = U/S TEL - R.L. of glacis (3)	Depth (y) (m) (4)	water level (5) = (2)+(4)	Ef = U/S TEL - R.L. of glacis (6)	Depth (y) (m) (7)	water level (8) =(2) + (7)
0	200	8.14	5	205	4.63	2.5	202.5
3	199	9.14	3.9	202.9	5.63	2	201
6	198	10.14	3.45	201.45	6.63	1.7	199.7
8.1*	197.3	10.84	3.25	200.55	7.33	1.6	198.9
9	197	11.14	3.15	200.15			
12	196	12.14	2.95	198.95			
13.17**	196.61	12.53	2.9	199.51			

Note: * Point at which hydraulic jump forms for pond level condition.

** Point at which hydraulic jump forms for high flood condition.

Post jump profile

From Table 5.9,

For high flood condition, with concentration and retrogression,

$Fr^2 = 6.55m$ and $y_1 = 2.90m$

For pond level condition, with concentration and retrogression,

$Fr^2 = 7.18m$ and $y_1 = 1.60m$

For determination of the post jump profile, different values of (x/y_1) are assumed and corresponding values of (y/y_1) are read from Annex V, for the known values of F_r^2. Calculations are shown in table 5.9. The distance x and y are measured from the point P where the hydraulic jump is formed.

Table 5.9: Post – jump profile calculations

High flood condition $y_1 = 2.90m$, $Fr^2 = 6.55$						Pond level condition $y_1 = 1.60m$ $Fr^2 = 7.18$				
S. No.	x/y1	y/y1	x	y	water level	x/y1	y/y1	x	y	water level
1	1	1.3	2.9	3.77	199.38	1	1.3	1.6	2.08	199.38
2	2.5	2	7.25	5.8	201.41	2.5	2	4	3.2	200.5
3	5	2.5	14.5	7.25	202.86	5	2.5	8	4	201.3
4	10	2.9	29	8.41	204.02	10	2.9	16	4.64	201.94
5	15	3	31.03	8.7	204.31	15	3	24	4.8	202.1
6						20	3.1	32	4.96	202.26

Step 10: Calculation of uplift pressure

Uplift pressure at various points can be calculated by plotting the hydraulic jump profile for the two conditions along with the subsoil HGL. Figure 5.10 (a) shows the plot for the high flood condition. Figure 5.10 (b) shows the plot for the pond level condition. From these plots, it is found that the maximum unbalanced head (H_d) in the jump trough is equal to 7.80m. The thickness of the glacis should be designed for 2/3 of this head (i.e. 5.2 m head) or for the static condition uplift pressure, whichever is greater. Figure 5.10 (c) shows the uplift pressure for no – flow condition (static condition).

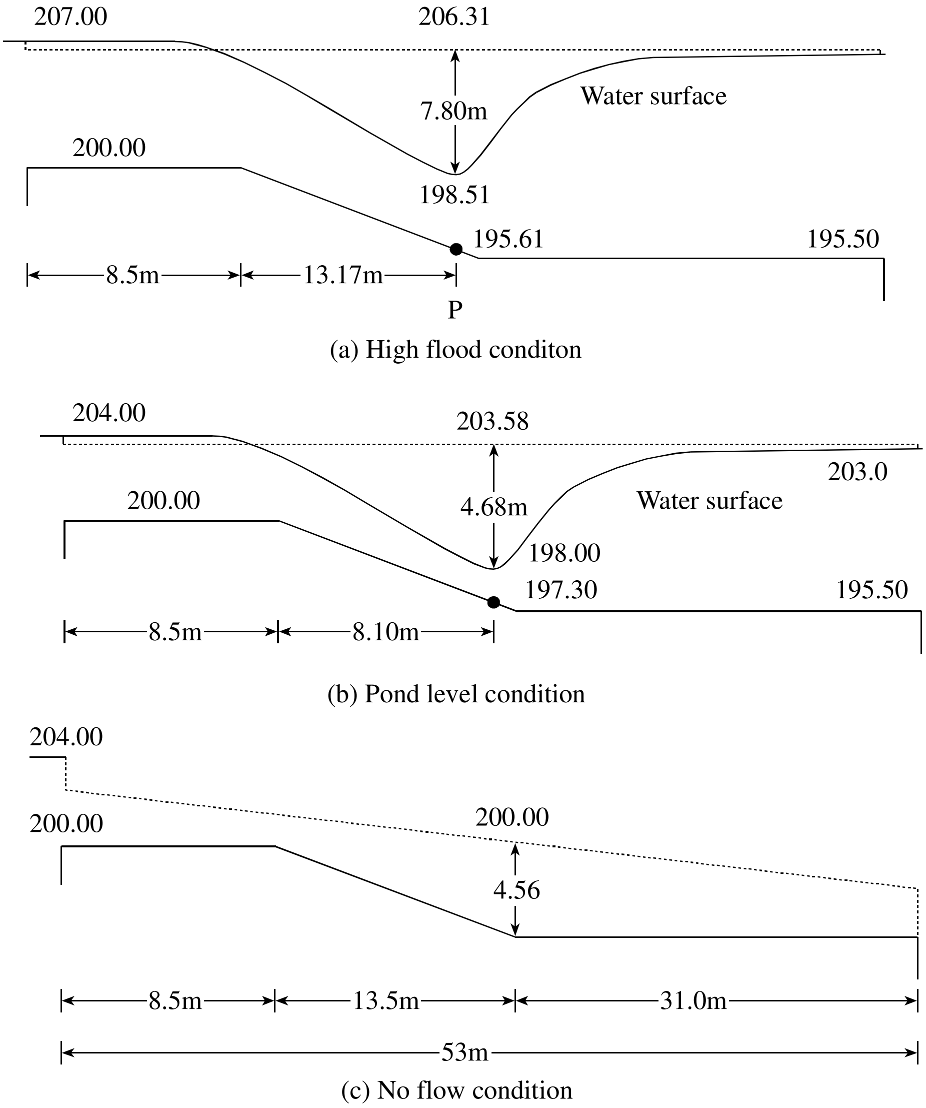


Figure 5.10: Jump profile

Table 5.10 gives the uplift pressure at various points under three conditions. The controlling uplift is taken the larger of the static condition and 2/3 of the dynamic condition.

It is seen that the dynamic uplift pressure governs the floor thickness up to a length of about 18.50m from the end of crest, whereas the static head governs the rest of the D/S floor thickness. Provide the floor thickness as shown in figure.

Table 5.10: Uplift pressure for different conditions

S.No.	Distance from end of crest (1)	No flow condition			High flood condition				Pond level condition				controlling uplift head (h) (13)	thickness t= h/g-1
		Level of glacis or floor (2)	HGL (3)	Unbalnce head (4) = (3) - (2)	water level (5)	HGL (6)	unbalance head hd (7) = (6) - (5)	2hd/3 (8)	water level (9)	HGL (10)	Unbalance head hd (11) = (10) - (9)	2hd/3 (11)		
1	0	200	200.92	0.92	205	206.46	1.46	0.97	202.5	203.64	1.14	0.76	0.97	0.78
2	3	199	200.73	1.73	202.9	206.42	3.52	2.35	201	203.37	2.37	1.58	2.35	1.89
3	6	198	200.54	2.54	201.45	206.39	4.94	3.29	199.7	203.59	3.89	2.59	3.29	2.66
4	8.1	197.3	200.4	3.1	200.56	206.36	5.8	3.87	198.9	203.58	4.68	3.12	3.87	3.12
5	12	196	200.16	4.16	198.95	206.32	7.37	4.91	200.51	203.55	3.04	2.03	4.91	3.96
6	13.17	195.61	200.09	4.48	198.51	206.31	7.8	5.20	200.71	203.54	2.83	1.89	5.20	4.19
7	13.5	195.5	200.06	4.56	198.7	206.3	7.6	5.07					5.07	4.09
8	18.5	195.5	199.75	4.25	200.4	206.25	5.85	3.90					4.25	3.43
9	23.5	195.5	199.43	3.93	201.6	206.19	4.59	3.06					3.93	3.17
10	28.5	195.5	199.11	3.61	202.86	206.14	3.28	2.19					3.61	2.91
11	33.5	195.5	198.8	3.3	203.58	206.08	2.5	1.67					3.30	2.66
12	38.5	195.5	198.46	2.96	203.91	206.03	2.12	1.41					2.96	2.39
13	44.5	195.5	198.1	2.6	204.31	205.96	1.65	1.10					2.60	2.10

Step 11: Protection Work

(a) D/S protection works

Normal scour depth R, as already calculated = 12.22m

Maximum scour = 2R = 24.44m

Scour level = 205.50 – 24.44 = 181.06 = 181.00

Scour depth below D/S bed, D_2 = 195.5 – 181.00 = 14.50m

(i) Inverted filter and concrete blocks

Length = $1.5D_2$ = 21.75m

Let us provide 17 rows of cement concrete blocks, 1.2m × 1.2m × 1.00m, having 10cm gaps filled with *bajri*. Total length = 17 × 1.2 +16 × 0.1 = 22.0m.

Provide 0.50m thick graded filter beneath the blocks.

Total thickness = 1.00 + 0.50 = 1.50m

(ii) Launching apron

Provide a launching apron of horizontal length equal to $1.5D_2 = 22.00m$.

Inclined length of launching apron = $D_2\sqrt{5} = 2.24D_2$

Let the thickness be 1m in a launched position.

Volume of stone in the launching apron = $2.24D_2$ m^3/m = 32.48 m^3/m

Thickness in the horizontal position = 32.48/22.00 = 1.48m. Provide 1.50m

(b) U/S protection works

(i) Concrete block

Maximum scour = 1.5R = 15 × 12.22 = 18.33m

Scour level = 207.00 – 18.33 = 188.67

Scour depth below U/S bed, D_1 = 200.00 – 188.67 = 11.33m

Length of concrete blocks D_1 = 11.33m

Let us provide 9 rows of cement concrete block 1.2m × 1.2m × 1m, having 10cm gaps filled with *bajri.*

Total length = 1.2 × 9 + 8 × 0.1 = 11.60m

Thickness of gravel pack below the concrete blocks = 0.50m

Total thickness = 1.00 + 0.50 = 1.50m

(ii) Launching apron

Provide a launching apron of horizontal length equal to $1.5D_1$ = 16.70m. Let us provide a length of 17.0m. Volume of stone = $2.24D_1$ = 25.38 m^3/m.

Thickness in the horizontal position = 25.38/17.0 = 1.50m

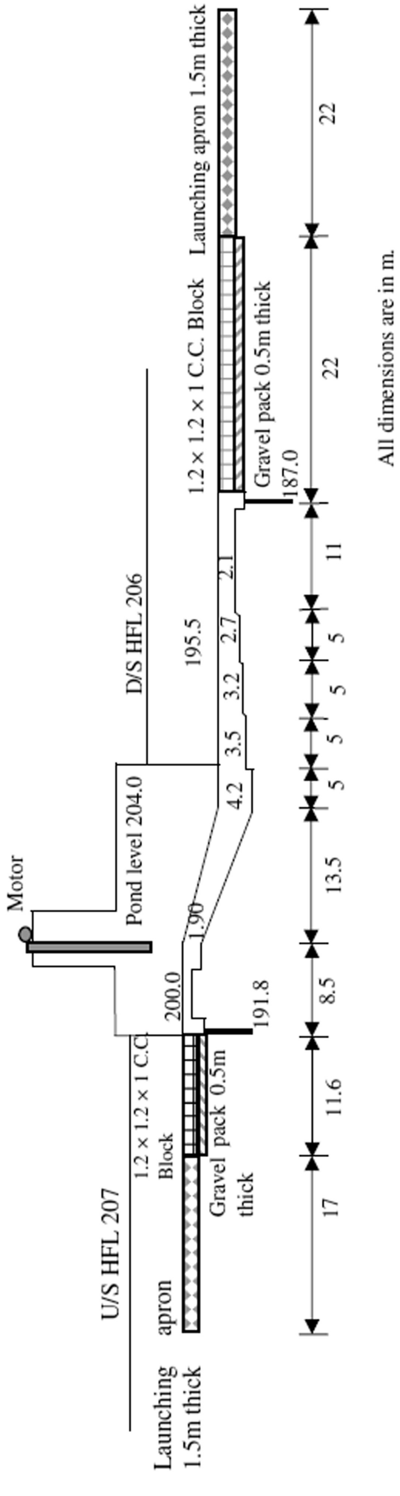


Figure 5.11: Designed cross section of under sluices

Example 5.6:

Design the weir bay section for the data given in example 5.5.

Solution:

Crest level of the weir bay section = 201.50.

Let the crest width = 2.0m

Step 1: Determination of discharge intensity (q) and head loss (H_L)

Let us find out the discharge intensity (q) and the head loss (H_L) for different conditions.

(a) High flood conditions

(b) Without flow concentration and retrogression

As B<2H//3, the weir act as a sharp – crested weir. H_e = 207.21 – 201.50 = 5.71m

$q = 1.84\ (H_e)^{3/2} = 25.11$ cumecs/m

U/S TEL = 207.21, D/S TEL = 206.21m

Head loss, H_L = 1.00mWith flow concentration and retrogression

$q = 1.2 \times 25.11 = 30.13$ cumecs/m

Head over the crest = $(30.13/1.84)^{2/3}$ = 6.45m

U/S TEL = 201.50 + 6.45 = 207.95 m

D/S TEL after retrogression = 206.21 – 0.5 = 205.71m

Head loss H_L= 207.95 – 205.71 = 2.24m

(c) Pond level conditions

(i) Without flow concentration and retrogression

U/S TEL, as calculated in example 5.4 = 204.10m

Head over the crest = 204.10 – 201.50 = 2.60m

$q = 1.84(2.60)^{3/2} = 7.71$ cumecs/m

D/S TEL = 203.60m

Head loss H_L = 204.10 – 203.60 = 0.50m

(ii) With flow concentration and retrogression

$q = 1.20 \times 7.71 = 9.25$cumecs/m

Head over the crest = $(9.25/1.84)^{2/3}$ = 2.93 m

U/S TEL = 201.50 + 2.93 = 204.43

D/S TEL = 203.10m

Head loss H_L = 204.43 – 203.10 = 1.33m

Step 2: Depth of sheet piles

Total discharge over the weir – bay section = 6843.61 m^3/sec

Overall water way = 300 + 48 = 348 m

Average discharge intensity, q = 6843.61/348 = 19.67 cumecs/m

Normal scour depth, $R = 1.35\ (q^2/f)^{1/3} = 1.35\ (19.67^2/1)^{1/3} = 9.84$ m

Bottom level of D/S pile = D/S water level after retrogression – 1.50R

= 205.50 – 1.5 × 9.84 = 190.74, say 190.50m

Bottom level of U/S pile = U/S water level – 1.25 R

= 207.00 – 1.25 × 9.84 = 194.70 say 194.50m

Depth of D/S file below floor = 197.00 – 190.50 = 6.50m

Depth of U/S pile below river bed = 200.00 – 194.50 = 5.50m

Step 3: Hydraulic jump calculation

S. No.	Item	High flood condition		Pond level condition	
		No concentration and retrogression	with concentration and retrogression	No concentration and retrogression	with concentration and retrogression
1	Discharge intensity (cumecs/m)	25.11	30.13	7.71	9.25
2	U/S water level	207	207	204	204
3	U/S TEL	207.21	207.95	204.1	204.43
4	D/S water level	206	205.5	203.5	203
5	D/S TEL	206.21	205.71	203.6	203.1
6	Head loss Hl	1	2.24	0.5	1.33
7	Ef2 from Blench curve	7.1	8.55	3.2	4.1
8	Level at which the hydraulic jump formed (5) - (7)	199.11	***197.16***	200.4	199
9	$Ef1 = Ef2 + H_L$	8.1	10.79	3.7	5.43
10	Pre jump Depth y1 from Montague's curve	2.4	2.3	1.1	1
11	Post jump depth y2 from Montague's curve	6.1	7.8	2.8	3.7
12	Length of horizontal floor = 5(y2 - y1)	18.5	***27.5***	8.5	13.5
13	Initial Froud no. $Fr = \frac{q}{\sqrt{gy1^2}}$	2.63	2.76	2.13	2.95
14	Fr^2	6.92	7.62	4.54	8.7
15	$y_c = \left(\frac{q2}{g}\right)^{1/3}$	4	4.52	1.82	2.06
16	$Z = H_L / y_c$	0.25	0.5	0.27	0.65

Lowest level of jump formation = 197.16m

Largest length of horizontal floor = 27.50 m

Let us keep the level and length of the D/S floor as 197.00 m and 28.0m, respectively.

Step 4: Length of floor

Maximum seepage head = pond level – D/S floor level

$H = 204.00 - 197.00 = 7.00m$

Exit gradient, $G_E = \frac{H}{d}\frac{1}{\pi\sqrt{\lambda}}$ or $\frac{1}{6} = \frac{7.00}{6.50}\frac{1}{\pi\sqrt{\lambda}}$ or $\lambda = 4.23$

Now, $\lambda = \frac{1+\sqrt{1+\alpha^2}}{2}$ or $\frac{1+\sqrt{1+\alpha^2}}{2} = 4.23$ or $\alpha = 7.39$, $b = 7.39 \times 6.5 = 48.05m$ say 50.0m.

Provide the floor lengths as follows

U/S glacis (1:1) = 1.5m

Crest width =2m

D/S glacis (3:1) = 13.50m

D/S horizontal floor = 28.00m

U/S horizontal floor = 5.00m

Total length = 50.00m

Step 5: Uplift pressure calculation

Let us assume the thickness of the upstream floor as 1.00m and the thickness at the end of the D/S floor as 2.00m.

For pile no 1	For pile no 2
$b = 50m$	$b = 50m$
$d = 5.5m$	$d = 6.5m$
$\alpha = b/d = 9.09$	$\alpha = b/d = 7.69$
$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = 5.07$	$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = 4.38$
$\Phi_{E1} = \frac{100}{\pi}\cos^{-1}\frac{0-\lambda}{\lambda} = 100\%$	$\Phi_{E2} = \frac{100}{\pi}\cos^{-1}\frac{\lambda - 2}{\lambda} = 31.71\%$
$\Phi_{D1} = \frac{100}{\pi}\cos^{-1}\frac{1-\lambda}{\lambda} = 79.66\%$	$\Phi_{D2} = \frac{100}{\pi}\cos^{-1}\frac{\lambda - 1}{\lambda} = 21.94\%$
$\Phi_{C1} = \frac{100}{\pi}\cos^{-1}\frac{2-\lambda}{\lambda} = 70.70\%$	$\Phi_{C2} = \frac{100}{\pi}\cos^{-1}\frac{\lambda - 0}{\lambda} = 0\%$
i. Correction for thickness for	i. Correction for thickness for
$\Phi_{C1} = \frac{79.66 - 70.70}{5.50} \times 1.00 = 1.63$	$\Phi_{E2} = \frac{31.71 - 21.94}{6.50} \times 2.00 = 3.00\%$
ii. Correction for mutual interference	ii. Correction for mutual interference
$= 19\left(\frac{d+D}{b}\right)\sqrt{\frac{D}{b'}}$	$= 19\left(\frac{d+D}{b}\right)\sqrt{\frac{D}{b'}}$
$= 19\left(\frac{4.5+8.5}{50}\right)\sqrt{\frac{8.5}{50}} = 2.04\%$	$= 19\left(\frac{4.50+0.5}{50}\right)\sqrt{\frac{0.5}{50}} = 0.19\%$
Corrected $\Phi_{C1} = 70.70 + 1.63 + 2.04 = \mathbf{74.37\%}$	Corrected $\Phi_{E2} = 31.71 - 3.00 - 0.19 = \mathbf{28.51\%}$

The levels of the sub – soil hydraulic gradient line (HGL) have been calculated in Table below.

S. No.	Flow Condition	U/S water level	D/S water level	Seepage Head	Height of elevation of subsoil HGL					
					U/S Pile			D/S pile		
					Φ_{E1}	Φ_{D1}	Φ_{C1}	Φ_{E2}	Φ_{D2}	Φ_{C2}
1	No flow	204	197	7	7	5.58	5.21	2	1.54	0
					204	202.58	202.21	199	198.54	197
2	High flood with flow concentration and retrogression	207	205.5	1.5	1.5	1.2	1.11	0.43	0.33	0
					207	206.7	206.61	205.93	205.83	205.5
3	Flow at pond level with flow concentration and retrogression	204	203	1	1	0.8	0.74	0.29	0.22	0
					204	203.8	203.74	203.29	203.22	203

For high flood condition and pod level condition, the suction pressure is computed from the hydraulic jump calculation given in table below.

Distance from the end of crest	RL of glacis	High flood condition			Pond level condition		
		U/S TEL = 207.95, q = 30.13 cumecs/m			U/S TEL = 204.43, q= 9.25 cumecs/m		
		Ef = U/S TEL - RL of glacis	Depth y (m)	Water level	Ef = U/S TEL - RL of glacis	Depth y (m)	Water level
0	201.5	6.45			2.93		
3	200.5	7.45	3.4	203.9	3.93	1.3	201.8
6	199.5	8.45	2.9	202.4	4.93	1.05	200.55
7.5*	199	8.95	2.7	201.7	5.43	1	200
9	198.5	9.45	2.6	201.1			
12	197.5	10.45	2.4	199.9			
13.02**	197.16	10.79	2.35	199.51			

Note *: Point at which hydraulic jump is formed for pond level condition.

**: Point at which hydraulic jump is formed for high flood condition.

The pre jump profile and post jump profile are calculate as per above example.

The maximum dynamic uplift pressure $h_d = 6.81$m, $2/3h_d = 4.54$m

The maximum static pressure = 3.80m.

Thickness at the toe of glacis = $\frac{4.54}{(2.24-1)} = 3.66$ say 3.70m.

This thickness shall be provided upto a distance of 5.50m from the toe of glacis. Static uplift pressure at the end of D/S floor = 199.00 – 197.00 = 2.00m.

Thickness at the end of D/S floor = $\frac{2}{(2.24-1)}$ = 1.61m say 1.7m

The thickness is reduces in step from 3.70 m to 1.70m.

Step 6: Protection works

(a) D/S protection works

(i) D/S concrete blocks

Normal scour depth R = 9.84m

Maximum scour depth = 2R = 19.68m

Scour level = 205.50 – 19.68 = 185.82

Depth D_2 below the D/S floor level = 197.00 – 185.82 = 11.18m

Length of inverted filter = $1.5D_2$ = 16.77m

Provide 13 rows of cement concrete blocks 1.2m × 1.2m × 1m, having 10cm gaps filled with *bajri*.

Total length = 13 × 1.2 + 12 × 0.1 = 16.80 m

Provide 0.50 m thick graded filter beneath the blocks.

Total thickness of the concrete block and filter = 1 + 0.5 = 1.50m.

(ii) Launching apron

Horizontal length of launching apron = $1.5D_2$ = 16.80m (say)

Volume of stone in the launching apron = $2.24D_2$ = 25.04m^3/m

Thickness of the apron in the horizontal position = 25.04/16.80 = 1.50m

(b) U/S protection works

(i) U/S concrete blocks

Normal scour depth R = 9.84m

Maximum scour depth = 1.5R = 14.67m

Scour level = 207.00 – 14.67 = 192.24

Depth D_1 below the U/S floor level = 200.00 – 192.24 = 7.76m

Length of concrete blocks = $1.5D_1$ = 11.6m

Provide 9 rows of concrete blocks of the size 1.2m × 1.2m × 1m, having 10cm gaps filled with *bajri*.

Total length = 9 × 1.2 + 8 × 0.1 = 11.60m

Thickness of gravel pack below the concrete blocks= 0.5m

Total thickness of concrete block and the gravel pack = 1.0 + 0.5 = 1.50m

(ii) U/S launching apron

Horizontal length of launching apron = $1.5D_1$ = 11.60m

Volume of stone = $2.24D_1$ = 2.24 × 7.76 = 17.38 m^3/m.

Thickness of apron in the horizontal position = 17.38/11.60 = 1.50m

Figure below shows the cross section of weir.

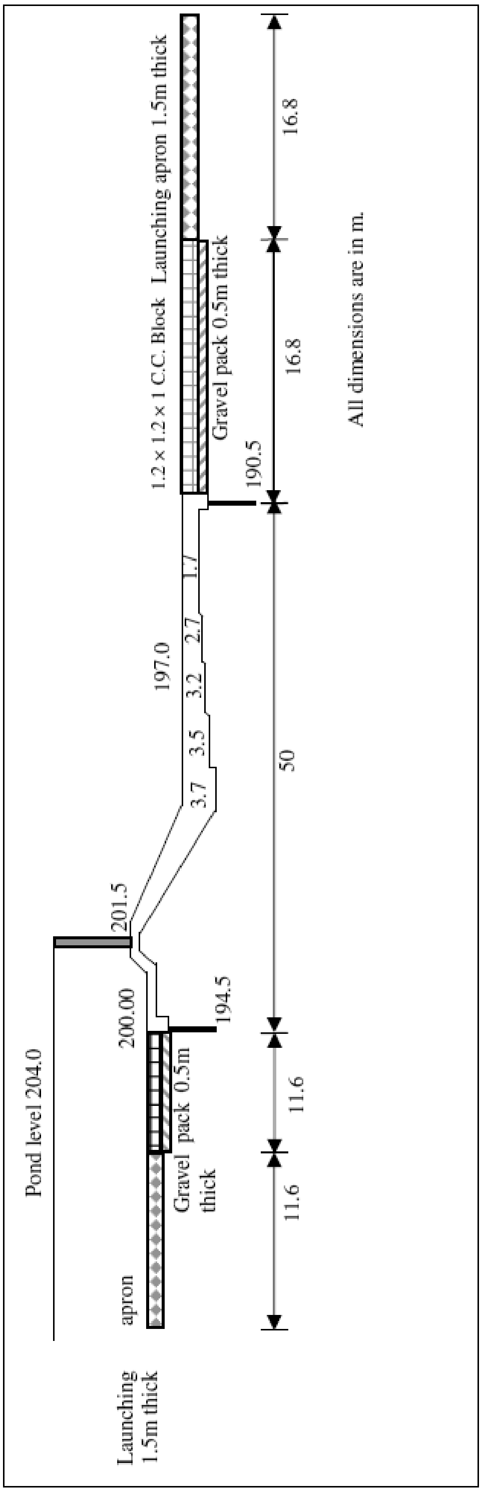


Figure 5.12: Designed section of the weir

5.6.2 Silt Excluder

The silt excluder is a device constructed in the river bed just upstream of the regulator to excludes silt from the water (source) entering the canal. It is so designed that the top and bottom layers of flow are separated with the least possible disturbance, the top sediment-free water being led towards the canal while the bottom sediment-laden water is discharged downstream of the diversion structure through under sluices. The device basically consists of a number of tunnels (Figure. 5.13) in the floor of the deep pocket of the river, isolated by a dividing wall. The sill or crest level of the regulator is kept the same as that of the top level of the roof slab of the tunnels.

The capacity of the tunnel is usually kept at about 20% of the canal discharge, and they are designed to maintain a minimum velocity of 2 – 3 m/sec (to avoid deposition in tunnels). (P.Novak., 2007)

The tunnel have bell – mouthed entrance so as to increase so as to increase the zone of suction. The radius of bell – mouthing varies between 2 – 6 times the tunnel width; the larger radius is for the tunnel away from the head regulator. The tunnels are throttled at the exit to increase the velocity to prevent silt deposition and to reduce the discharge. The roof of the tunnels must be designed to withstand the full water load up to the high flood level, with no water inside. The tunnels should be safe against impact by debris, boulders shingles etc.

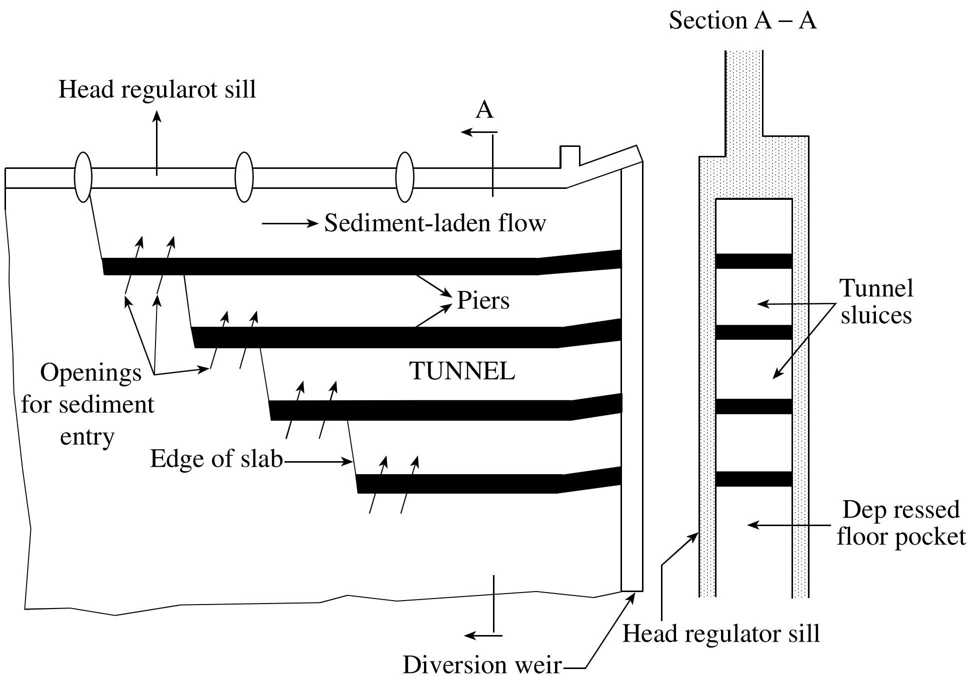


Figure 5.13: Silt excluder under tunnel type

Design consideration

It has been found from model studies that if the discharge in the silt excluder is restricted to about 15% to 20% of the canal discharge, satisfactory silt exclusion can be achieved. In order to keep the tunnels free from sediments, a minimum velocity of 2 m /sec for river with sand particles and 4.5 m/sec for river with boulders, should be maintained through the excluder. The cross – sectional area (A) of the tunnels may be determined from the discharge and the velocity. Thus

$$\text{Area of flow (A)} = \frac{\text{Discharge}}{\text{Velocity}}$$

The height (h) of the tunnel is given by, h = crest level of head regulator – upstream floor level of the undersluice – thickness of the roof slab.

The height (h) generally varies from 0.5 m to 0.6m for rivers with sand particles and 0.8m to 1.2m for rivers with boulder.

Knowing the total area of cross – section (A) of the tunnels and the height (h), the clear width can be found. Thus B = A/h

The clear width (B) is suitably divided into a number of tunnels such that the span of the roof slab is not very large and a suitable number of complete tunnels, and their divide walls, can be accommodated in each undersluices opening.

The difference of upstream and downstream water levels during floods (i.e. afflux) is usually between 0.6 m to 1m, which is sufficient for the silt excluder to work satisfactorily. However, it is preferable to provide more head than the minimum required head in order to pass extra discharge for better flushing. To ensure that the silt – laden water is promptly removed through the silt excluder, sometimes a separate outfall channels is provided on the downstream of the excluder, especially in rivers with gravel and boulders. For closing the tunnels, grooves are provided at their entrances in which temporary gates are inserted whenever required.

Example 5.7:

Design a silt excluder for the driven head works for the following data:

Full supply discharge of canal = 200 m^3/sec

Crest level of undersluices = 200.00m

Crest level of head regulator = 202.00m

Bay width of undersluices = 16m

Solution:

Let us take the design discharge as 20% of that of canal design discharge.

Q = 0.2 × 200 = 40 m^3/sec

Let us assume a velocity of 2 m/sec,

Area of cross – section = 40/2 = 20m^2

Let us assume the thickness of roof slab as 0.2m

Height of tunnel = 202.00 – 0.2 – 200.00 = 1.80m.

Total clear width = 20/1.80 = 11.11 m

Provide 6 no of tunnel, hence each tunnel width = 11.11/6 = 1.85m provide 2 m width each.

Let thickness of dividing wall = 0.75m

Overall width = 6 × 2 + 5 × 0.75 = 15.75 say 16.0m

5.7 Silt Ejector or Extractor

The silt ejector is a device constructed on the canal downstream of the head regulator but upstream of the settling basin (if any), by which the silt, after it has entered the canal, is extracted.

1. Vane type ejector

The layout of a vane type ejector is shown in Figure 5.14. A diaphragm at the canal bed separates the top layers from the bottom ones. On entering the depressed area of the canal bed, the bottom sediment laden layer are diverted by the curved vanes towards the escape chamber. The design should be such that the entry disturbance are minimal; the streamlined vane passages accelerate the flow through them, thus avoiding deposition.

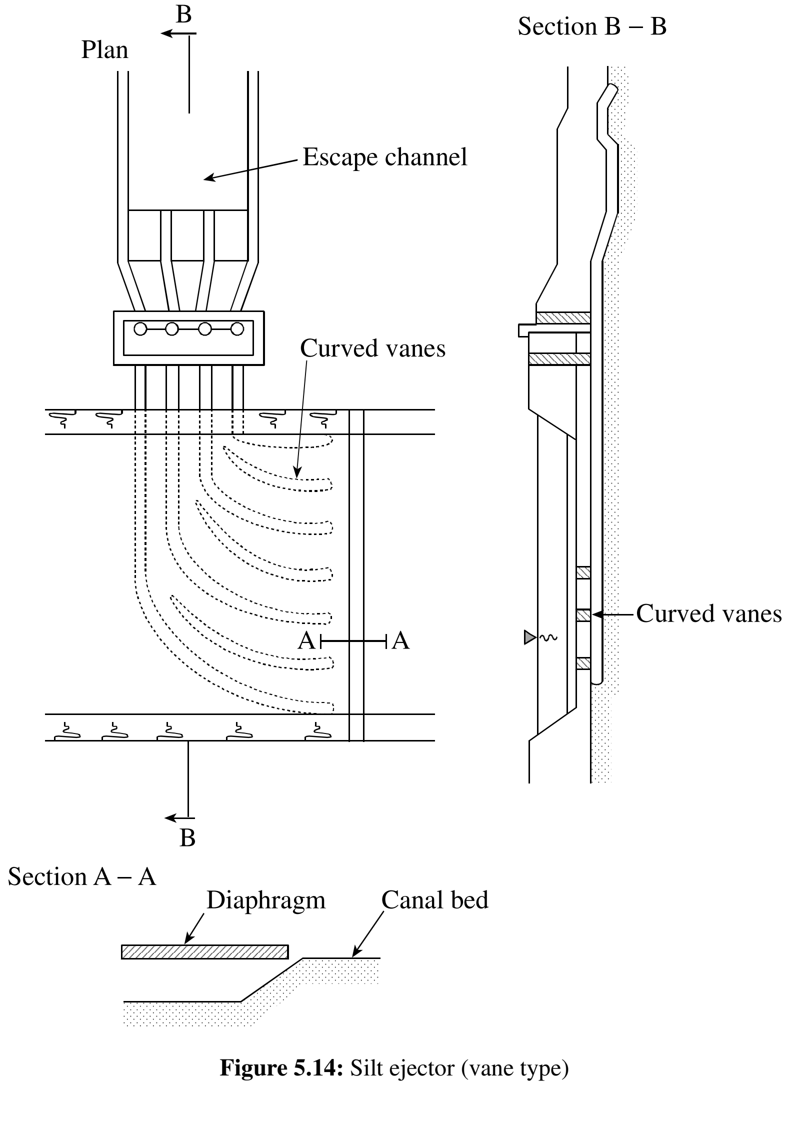


Figure 5.14: Silt ejector (vane type)

2. Vortex type ejector

The vortex tube ejector (Figure 5.15) consist of pipe with a silt along its top, places across the bottom of the canal at an angle of around $30^0 - 90^0$ to the direction of flow. The vortex motion within the tube draws the sediment into it, and the wall velocities along the tube eventually eject the sediment at its discharge end. A properly designed vortex tube ejector can be more efficient than any other conventional ejector, with less water loss.

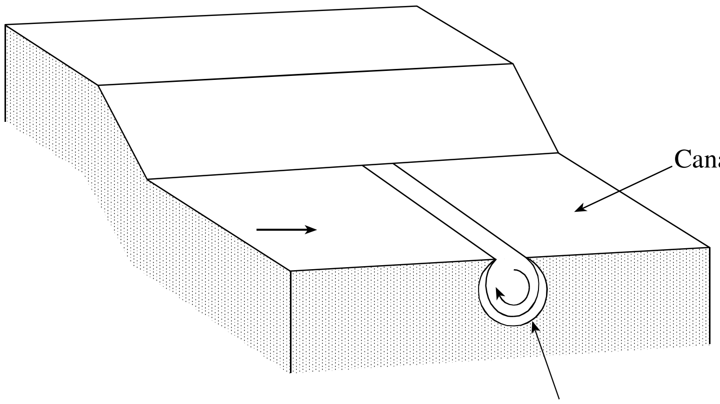


Figure 5.15: Silt ejector vortex type

Table 5.11: Differentiate between Silt Excluder and Silt Ejector

S.N.	Silt Excluder	Silt Extractor
1.	The silt excluder is located upstream of undersluices between head regulator and divide wall for excluding the bed load.	The silt extractor or ejector is located in canal at some distance downstream of head regulator for ejecting the suspended load to river.
2.	The structure is heavy as it is subjected to large forces.	The structure is relatively light.
3.	Silt exclusion can be done only once before the water enters the canal.	Silt extraction can be done a number of times by installing various extractors on the canal.
4.	Good approach conditions are difficult to achieve.	Good approach conditions can be easily achieved.
5.	The capacity of head regulator and canal is not to be increased because of a silt excluder.	The capacity of head regulator and canal has to be increased to carry extra discharge for the silt extractor.
6.	Working head is always available at the excluder.	Working head is reduced when the canal supply is low.
7.	The tunnels are straight, quite large and are not liable to be clogged.	The tunnels are curvilinear small and may be choked by debris.
8.	The overall cost is high.	The overall cost is low if the extractor is not very far off from the head regulator.

5.8 Design of Head Regulator

A head regulator is provided at the entrance to the off taking main canal at the diversion headworks. It is located on the bank of the river just upstream or the undersluice pocket.

1. Crest level

The crest level of the head regulator is usually kept 1.25 to 1.50m higher than the crest level of the undersluices. However, if a silt excluder is provided in the undersluices section, the crest level of the head regulator is kept 1.75 to 2.25m higher that of the undersluices.

2. Discharge formula

(a) Pond level condition

The head regulator is usually provided with a very wide and shallow waterway. The drowned weir formula is generally used for computing the discharge. Figure 5.16 (a)

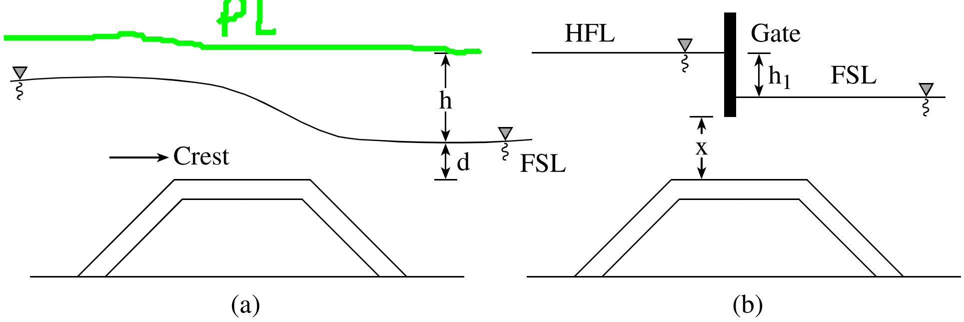


Figure 5.16: Flow condition at head regulator

$$Q = \frac{2}{3} Cd_1 \times B \times \sqrt{2g} \times [(h + h_a)^{3/2} - h_a^{3/2}] + Cd_2 \times B \times d \times \sqrt{2g \times (h+h_a)} \quad \ldots 5.10$$

Where Cd_1 and Cd_2 are the discharge coefficients for the free portion and drowned portions, respectively. Their values are generally taken as 0.577 and 0.80 respectively.

h= head causing flow = pond level – FSL

d = depth of water over crest on the D/S

B = length of crest and

h_a = head due to velocity approach.

Neglecting velocity head (small quantity) and solving by using $C_{d1} = 0.577$ and $C_{d2} = 0.80$, we get

$$Q = 1.71 \times B \times h^{3/2} + 3.54 \times B \times d \times h^{1/2}$$

(b) High flood conditions

During high floods, the water level on the upstream rises and the regulator gates are partly closed. Let *x* be the height of the gate opening when the canal takes the full supply discharge during floods Figure 5.16 (b). The submerged orifice formula is used for the discharge. Thus

$$Q = C_d A \sqrt{2gh_1} \quad \ldots 5.11$$

Where h_1 is the head causing flow= U/S HFL = FSL

C_d = discharge coefficient (0.62)

A = area of flow

$$\text{Therefore } Q = 0.62 \times x \times B \times \sqrt{2gh_1} \quad \ldots 5.12$$

The value of *x* can be determined from the known values of Q, B and h_1.

3. Discharge intensity and loss of head

(a) Pond level conditions

Discharge intensity, $q = Q/B$

Loss of head H_L = Pond level – FSL

(b) High flood conditions

Velocity through gates, $V = \dfrac{Q}{xB}$

Velocity head, $h_d = \dfrac{v^2}{2g}$

Loss of head at entry $= 0.5\dfrac{v^2}{2g}$

TEL just upstream at gates= U/S HFL + velocity head in the river

TEL just downstream of gates = TEL just U/S of gate $- 0.5\dfrac{v^2}{2g}$

Loss of head H_L = TEL just D/S of gate – FSL

Discharge intensity, $q = Q/B$

4. Depth of piles

(a) U/S pile

The depth of U/S pile is kept the same as in the case of the undersluices.

(b) D/S pile

The bottom level of the D/S pile is taken at a depth of 1.25 R to 1.50 R below FSL.

5. Hydraulic jump calculations

Hydraulic jump calculations are done for the pond level condition and the high flood condition. The level at which the hydraulic jump is formed is located and the length of jump is determined, as in the case of the undersluices section. The level of the D/S floor is fixed below the lowest level at which the hydraulic jump is formed.

6. Thickness of impervious floor

The uplift pressures are determined at the key points of the upstream and downstream piles for the following conditions:

- No flow conditions
- Pond level flow condition
- High flood flow condition

Thickness of the impervious floor is determined for the worst case.

7. Protection works

(a) U/S protection work

The upstream protection works are similar to those in the case of undersluices.

(b) D/S protection work

The maximum scour depth is taken as 2R. The length of inverted filter is taken equal to the depth d_2 of the pile below the downstream bed.

The concrete blocks 1.25m × 1.25m × 1.0m with 10cm gaps filled with *bajri*, are usually provided over 0.8cm thick invert filter. Thus a total thickness of 1.80m is provided.

The horizontal length of the D/S launching apron is obtained from the volume of stone equal to $2.24d_2$, and a thickness equal to 1.80m in the horizontal position.

Example 5.8:

Design a head regulator for the data given below:

Crest level of under sluices = 200.00m

Pond level = 204.00m

U/S high flood level = 207.00m

Head due to velocity of approach = 0.21m

Full supply discharge = 200 m^3/sec

Full supply level = 203.00m

Bed level of canal = 199.50m

A silt excluder is provided in the undersluices. Take $G_E = 1/6$

Solution:

Crest level of the head regulator = crest level of undersluices + 2.0m

= 200.00 + 2.00 = 202.00m

From equation 5.10,

$$Q = \frac{2}{3} Cd_1 \times B \times \sqrt{2g} \times [(h + h_a)^{3/2} - h_a^{3/2}] + Cd_2 \times B \times d \times \sqrt{2g \times (h+h_a)}$$

Where h = pond level – FSL = 204.00 – 203.00 = 1.00m

d = 203.00 – 202.00 = 1.00m

Taking $Cd_1 = 0.577$, $Cd_2 = 0.80$ and neglecting the velocity of approach.

$$200 = \frac{2}{3} \times 0.577 \times 4.43 \times B \times (1)^{3/2} + 0.80 \times B \times 1 \times \sqrt{2g \times 1}$$

200 = (1.704 + 3.544) B or B = 38.11m say 40m.

Provide 5 bays of 8.0m each, with a clear waterway of 40.00m.

Provide 4 piers each of 1.5m thickness.

Overall waterway =5 × 8 + 4 × 1.5 = 46m

At pond level, loss of head, H_L = 204.00 – 203.00 = 1.00m

Discharge intensity, q = 200/40 = 5 cumecs/m

At high flood level, let *x* be the height of gate opening.

From equation 5.12, $200 = 0.62\ x \times x\ 40\sqrt{2 \times 9.81 \times 4.0}$

$x = 0.91\text{m}$

Velocity through opening, $V = \dfrac{200}{0.91 \times 40} = 5.49$ m/sec.

Loss of head at entry $= 0.5\dfrac{v^2}{2g} = 0.77\text{m}$

TEL just upstream of gate = 207.00 + 0.21 = 207.21m

TEL just D/S of gate = 207.21 – 0.77 = 206.44m

Head loss H_L = 206.44 – 203.00 = 3.44m

Discharge intensity q = 5 cumecs/m

Hydraulic jump calculation

The calculations are given in table below:

S. No.	Item	High flood condition	Pond level condition
1	Discharge intensity q (cumecs/m)	5	5
2	U/S water level (m)	207	204
3	D/S water level (m)	203	203
4	U/S TEL just upstream of gate (m)	206.44	204
5	D/S TEL (m)	203	203
6	Loss of head (HL)	3.44	1
7	D/S specific energy, Ef2 (from blench curve) (m)	3.3	2.65
8	U/S specific energy, Ef1 = Ef2 + H_L (m)	6.74	3.65
9	Pre jump depth y1 from Montague curve (m)	0.45	0.65
10	Post jump depth y2 from Montague curve (m)	3.1	2.5
11	level at which hydraulic jump is formed = (5) - (7) (m)	***199.7***	200.35
12	Length of impervious floor = 5(y2 – y1) (m)	***13.25***	9.25
13	Froud no. $Fr = \dfrac{q}{\sqrt{gy^3}}$	5.29	3.05
14	Fr^2	27.98	9.3

Provide D/S floor at 199.50. Length of D/S floor = 13.50m

Depth of sheet pile

(a) U/S pile

Bottom level of U/S pile will be kept at an elevation of 191.80m as in the case of undersluices.

Depth of U/S pile below the bed = 200.00 – 191.80= 8.20m

(b) D/S pile

Discharge intensity q = 5.0 cumecs/m

Scour depth $R = 1.35\left(\frac{q^2}{f}\right)^{\frac{1}{3}} = 1.35\left(\frac{5^2}{1}\right)^{\frac{1}{3}} = 3.95$m

Maximum scour depth = 1.5R = 5.93m

Bottom level of D/S pile = D/S water level – 5.93 = 203.00 – 5.93 = 197.07 m

Depth of D/S below bed, d_2 = 199.50 – 197.07 = 2.43m

Provide d_2 = 5.00m to reduce exit gradient.

Therefore, bottom level of D/S pile = 199.50 – 5.00 = 194.50m

Length of impervious floor

Seepage head (H) = 207.00 – 199.50 = 7.50m

Now, $G_E = \frac{H}{d}\frac{1}{\pi\sqrt{\lambda}}$ or $\frac{1}{6} = \frac{7.5}{5}\frac{1}{\pi\sqrt{\lambda}}$ or $\lambda = 8.20$

Now $\frac{1+\sqrt{1+\alpha^2}}{2} = 8.20$ or $\alpha = 15.36$, $\alpha = b/d = 15.36$

b = 15.36 × 5 = 76.36m say 77m.

Provide the length as follows:

Length of U/S floor = 54m.

Width of crest = 2.00m

D/S glacis = 7.50m

D/S floor = 13.50m

Total = 77.00m

Uplift pressure calculation

Let us assume the thickness of U/S floor as 1.0m and that at the end of D/S floor as 1.50m

(a) U/S pile

d = 8.20m, b = 77m, $\alpha = 9.39$, $\lambda = 5.22$

$$\Phi_{E1} = \frac{100}{\pi}\cos^{-1}\frac{0-\lambda}{\lambda} = 100\%$$

$$\Phi_{D1} = \frac{100}{\pi}\cos^{-1}\frac{1-\lambda}{\lambda} = 79.97\%$$

$$\Phi_{C1} = \frac{100}{\pi}\cos^{-1}\frac{2-\lambda}{\lambda} = 71.16\%$$

(i) Correction for thickness for $\Phi_{C1} = \frac{79.97 - 71.16}{8.20} \times 1.00 = 1.07\%$

(ii) Correction for mutual interference $= 19\left(\frac{d+D}{b}\right)\sqrt{\frac{D}{b'}} = 19\left(\frac{7.2+4.5}{77}\right)\sqrt{\frac{4.5}{77}} = 0.70\%$

Corrected Φ_{C1} = 71.16 + 1.07 + 0.70 = **72.93%**

(b) D/S pile

d = 5.0m, b = 77m, α = 15.4, λ=8.22

$$\Phi_{E2} = \frac{100}{\pi}\cos^{-1}\frac{\lambda-2}{\lambda} = 22.68\%$$

$$\Phi_{D2} = \frac{100}{\pi}\cos^{-1}\frac{\lambda-1}{\lambda} = 15.86\%$$

$$\Phi_{C2} = \frac{100}{\pi}\cos^{-1}\frac{\lambda-0}{\lambda} = 0\%$$

(i) Correction for thickness for $\Phi_{E2} = \frac{22.68-15.86}{5} \times 1.50 = 2.05\%$

(ii) Correction for mutual interference $= 19\left(\frac{d+D}{b}\right)\sqrt{\frac{D}{b'}} = 19\left(\frac{3.5+6.2}{77}\right)\sqrt{\frac{6.2}{77}} = 0.68\%$

Corrected Φ_{E2} = 22.68 – 2.05 – 0.68 = **19.95%**

Height and elevation of the subsoil HGL are calculated in the table below.

S. No.	Flow Condition	U/S water level	D/S water level	Seepage Head	Height of elevation of subsoil HGL					
					U/S Pile			D/S pile		
					Φ_{E1}	Φ_{D1}	Φ_{C1}	Φ_{E1}	Φ_{D1}	Φ_{C1}
1	No flow	207.00	199.50	7.50	7.50	6.00	5.47	1.50	1.19	0.00
				RL	207.00	205.50	204.97	201.00	200.69	199.50
2	High flood level on U/S	207.00	203.00	4.00	4.00	3.20	2.92	0.80	0.64	0.00
				RL	207.00	206.20	205.92	203.80	203.64	203.00
3	Pond level on Upstream	204.00	203.00	1.00	1.00	0.80	0.73	0.20	0.16	0.00
				RL	204.00	203.80	203.73	203.20	203.16	203.00

The uplift pressures are the maximum for the no flow condition.

For no flow condition, residual head at D/S of floor = 1.5m

Thickness of floor $= \frac{1.5}{2.24-1.00} = 1.21$m, adopt 1.50m

At the end of the D/S glacis

Residual head $= 1.50 + \frac{5.47-1.50}{77} \times 13.5 = 2.20$m

Thickness of floor $= \frac{2.20}{2.24-1.00} = 1.77$m provide 2.0m

Protection works

(a) U/S protection works

They shall be same as in the case of the undersluice section.

(b) D/S protection works

Normal scour depth R = $1.35(q^2/f)^{1/3} = 3.95$m

Anticipated scour = 2R = 7.90m

Downstream scour level = 203.00 – 7.90 = 195.10

Depth d_2 below the D/S floor = 199.50 – 195.10 = 4.40m. Take d_2 as 5.0m

(i) Block protection

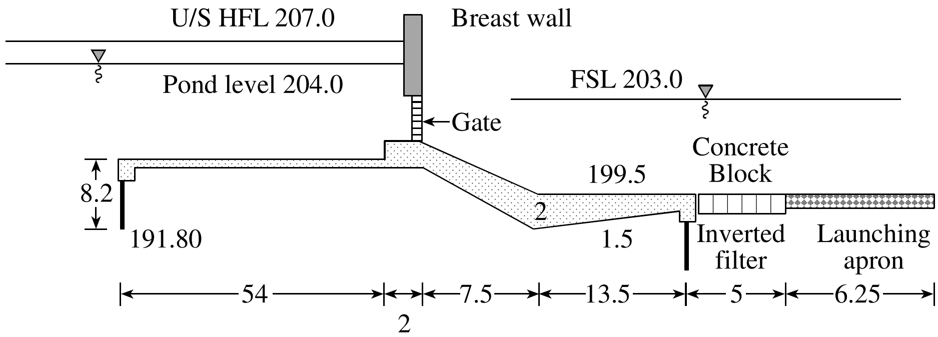
The length of the inverted filter = d_2 = 5.0m. Provide 1.25m × 1.25m × 1.00m thick concrete block, with 10cm gaps filled with *bajri*, over 0.8 thick graded filter. Thus a total thickness of 1.80m is provided.

(ii) Launching apron

Thickness of the launching apron = 1.80m

Volume of stone = $2.24d_2 = 11.2\ m^3/m$

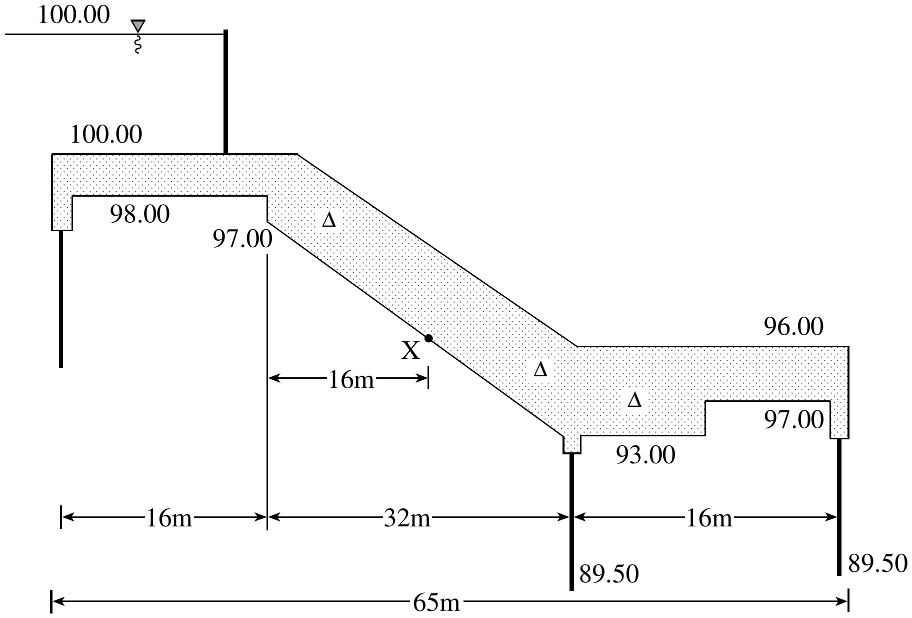
Length of apron required = 11.2/1.80 = 6.22m, say 6.25m.



All dimensions are in m.

Example 5.9:

The accompanying figure shows the profile of a weir. The various levels shown in the figure are in meters. Determine the uplift pressures at the key points and the exit gradient and find whether the section provided is safe against piping if it is founded on fine sand with permissible exit gradient of 1/6. Also find uplift pressure at point X and check whether it is safe against uplift.



Solution:

For pile no 1	For pile no 2	For pile no 2
b = 65m d = 100 – 91 = 9m 1/α = b/d = 0.14 From Khosla's Curve Φ_{D1}= 77% Φ_{C1}= 67% i. Correction for thickness for Φ_{C1}= $\frac{77-67}{5.9} \times 1.00 = 2.2$ ii. Correction for mutual interference $= 19\left(\frac{d+D}{b}\right)\sqrt{\frac{D}{b'}}$ $= 19\left(\frac{7+8.5}{65}\right)\sqrt{\frac{8.5}{48}} = 1.9\%$ Corrected Φ_{C1} = 67 + 2.2 + 1.9 = **71.1%**	b_1= 48m b=65m d=96 – 89.5 = 6.5m α_1= b/d= 65/6.5=10 b_1/b = 48/65=0.746 From Khosla's curve Φ_{E2} = 41% Φ_{D2}= 34% Φ_{C2}= 26%	b = 65m d = 6.5m 1/α = d/b= 0.1 From the Khosla's curve Φ_{E3} = 29% Φ_{D3} = 20% i. Correction for thickness for Φ_{E3}= $\frac{29-20}{6.5} \times 2.00 = 2.8\%$ ii. Correction for mutual interference of pile no. 2 = $19\left(\frac{d+D}{b}\right)\sqrt{\frac{D}{b'}}$ = $19\left(\frac{4.50+4.5}{65}\right)\sqrt{\frac{4.5}{16}}$ = 1.4% Corrected Φ_{E2} = 29 – 2.8 – 1.4 = **24.8%**

(i) Correction for floor thickness for $\Phi_{E2} = \frac{41-34}{6.5} \times 3.00 = 3.2\%$

(ii) Correction for floor thickness for $\Phi_{C2} = \frac{34-26}{6.5} \times 3.00 = 3.7\%$

(ii) Correction for mutual interference of pile no 1 for $\Phi_{E2} = 19\left(\frac{d+D}{b}\right)\sqrt{\frac{D}{b'}} = 19\left(\frac{3.5+2}{65}\right)\sqrt{\frac{2}{48}} = -0.3\%$

(iii) Correction for mutual interference of pile no 3 for $\Phi_{C2} = 19\left(\frac{d+D}{b}\right)\sqrt{\frac{D}{b'}} = 19\left(\frac{3.5+3.5}{65}\right)\sqrt{\frac{3.5}{16}} = 1\%$

(iv) Correction due to slope for Φ_{E2}

From table 5.4, for a slope of 1 in 8 the correction % of pressure = 2.0% and hence

Correction $= 2.0 \times \frac{b_s}{b'}$

$b_s = 32m$ and $b' = 48m$

Correction $= 2.0 \times \frac{32}{48} = 1.3\%$

Hence, corrected value of Φ_{C2} and Φ_{E2} are as follows

$\Phi_{E2} = 41 - 3.2 - 0.3 + 1.3 = 38.8\%$

$\Phi_{C2} = 26 + 3.7 + 1.0 = 30.7\%$

Maximum seepage head = 103 – 96 = 7.0m

The percentage pressure and the residual pressure head at key points are tabulated below.

Point	% pressure	Residual pressure Head (m)
D_1	77.0	5.39
C_1	71.1	4.98
E_2	38.8	2.72
D_2	34.0	2.38
C_2	30.7	2.15
E_3	24.8	1.74
D_3	20.0	1.40

Exit Gradient

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

$H = 103 - 96 = 7m$

$d = 96 - 89.5 = 6.5m$

$\alpha = b/d = 65/6.5 = 10$

From exit gradient curve for $\alpha = 10$

$$\frac{1}{\pi\sqrt{\lambda}} = 0.135$$

G_E = 1/6.85 which is less than permissible exit gradient 1/6 hence safe.

Uplift pressure at X is $P_x = P_{c1} - \left(\frac{P_{c1} - P_{E2}}{48}\right) \times 32$

$$P_x = 4.98 - \left(\frac{4.98 - 2.72}{48}\right) \times 32 = 3.47\text{m}$$

The floor thickness required at X = $\frac{P_x}{(G-1)}$ = 2.8m

Actual thickness provided = 3m

Hence it is safe against uplift.

Example 5.10:

An irrigation barrage has to be designed to pass a flood of 10,000 m^3/sec, through alluvium media (mean diameter of particles = 0.33mm). The flood level, pond level and downstream floor level are 207.0m, 204.0m and 198.0m respectively. If the safe exit gradient is 1/6, compute minimum total impervious floor length required to safeguard the structure from piping. Prepare a conceptual section of the designed structure.

Solution:

Given, High flood discharge Q = 10000 m^3/sec

d_{50} = 0.33mm

HFL before construction = D/S HFL = 207.0 m

Pond level = 204.0 m

D/S bed level = 198.0 m

Length of water way P = $4.75\sqrt{Q}$ = 475 m (for alluvial rivers)

Flood intensity q= Q/L = 10000/475= 21.05 cumecs/m

Normal scour depth R = $1.35\left(\frac{q^2}{f}\right)^{\frac{1}{3}}$

Where f = $1.76\sqrt{d_{mm}}$ = 1.01

Now R = 10.25 m

Let us take maximum scour depth at D/S = 1.5R = 15.375m

Bottom level of D/S pile = 207 – 15.375 = 191.625m

Depth of D/S pile = D/S bed level – Bottom level of D/S pile = 198 – 191.625 = 6.375m

We have; Exit Gradient $G_E = \frac{H}{d}\frac{1}{\pi\sqrt{\lambda}}$

H = Pond level – D/S floor level= 6m

d = 6.375m

Now, $1/6 = \frac{6}{6.375} \frac{1}{\pi\sqrt{\lambda}}$

$$\frac{1}{\pi\sqrt{\lambda}} = 0.177$$

$$\lambda = 3.234$$

We have $\lambda = \frac{1+\sqrt{1+\alpha^2}}{2}$

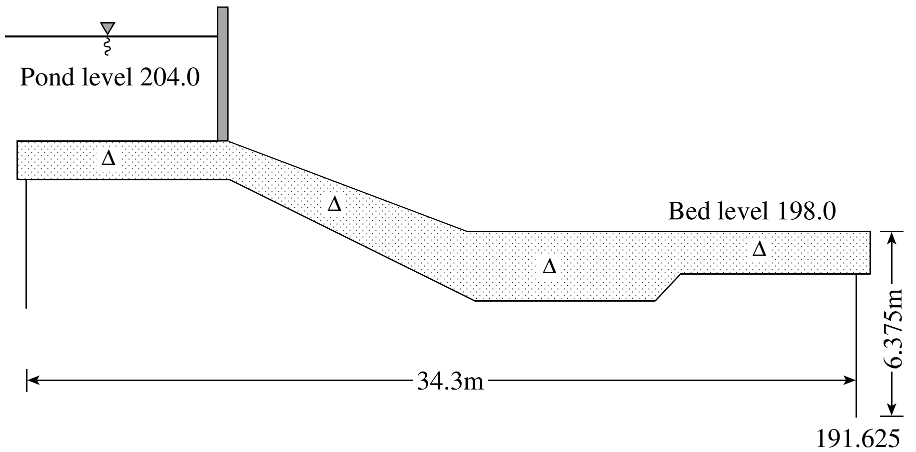
$$3.234 = \frac{1+\sqrt{1+\alpha^2}}{2}$$

Now $\alpha = 5.375$

Again $\alpha = b/d$

Now $b = \alpha \times d = 5.375 \times 6.375 = 34.265\text{m} \approx 34.3\text{m}$

Hence minimum total impervious floor length required to safe guard the structure from piping = 34.3 m.



Example 5.11:

A river discharge 2000 cumecs of water at high flood level of 273 m. A weir is constructed for flow diversion with a crest length of 150m and total length of concrete floor as 50m. The weir has to sustain the under seepage at a maximum static head of 2.4m. The silt factor and the safe exit gradient for the river bed material are 1.1 and 1/6 respectively. Determine the depth of cutoff required at the downstream end of the concrete floor. Take the level of downstream concrete floor level 270m. Check for exit gradient.

Solution:

Given, Q = 2000 cumecs

Length weir, L = 150m

Silt factor, f = 1.7

Now, discharge per unit length, $q = \frac{Q}{L} = \frac{200}{150} = 13.33$ cumecs/m

Scour depth, $R = 1.35\left(\frac{q^2}{f}\right)^{1/3} = 7.35\text{m}$

HFL before construction or D/S HFL = 273m

Now, R.L. of D/S cutoff = D/S HFL – 1.5R = 273 – 1.5 × 7.35 = 261.975m

R.L of D/S concrete floor = 270m

Depth D/S cutoff =270 – 261.975 = 8.03m

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

Here, H_s = 2.4m

d = D/S cutoff depth = 8.03m

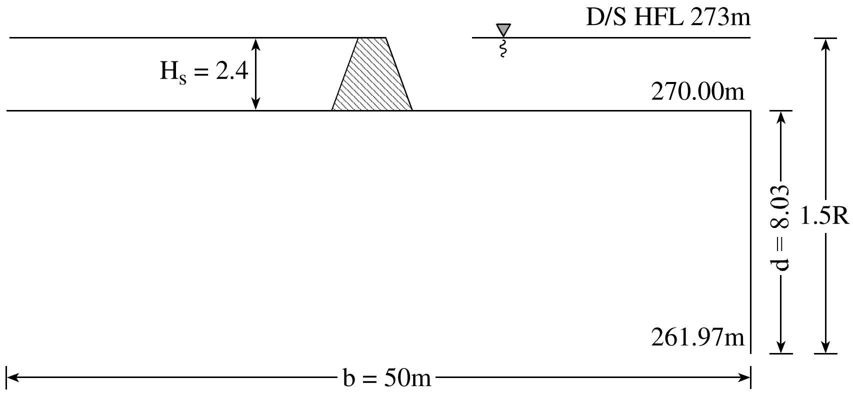
Length of concrete floor, b = 50m

Now, $\alpha = \frac{b}{d} = 6.23\text{m}$

$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = 3.654$

Now, $G_E = \frac{2.4}{8.03} \times \frac{1}{\pi\sqrt{3.65}} = \frac{1}{20.08} < \frac{1}{6}$

Hence, the weir floor is safe against piping.



Example 5.12:

A river carries a high flood discharge of 16000m^3/sec with its average bed level at 200.0m. A canal carrying 200m^3/sec is to take off from the headworks. The full supply level of the canal at its head is 203.0m. The high flood level before construction is 205.7m and Lacey's silt factor is equal to unity. Fix suitable values for the water way and crest levels of weir, under sluices and canal head regulator. Assume suitably any other data if required.

Solution:

Design discharge for under sluices is taken maximum of:

- 20 % of high flood = 3200 m^3/sec
- 2 times the canal design flow = 2 × 200 = 400 m^3/sec

Hence design discharge for under sluices = 3200 m^3/sec.

Remaining flood discharge of (16000 – 3200 = 12800 m^3/sec) will be passed through weir

Weir Portion

Design discharge = 12800 m^3/sec

Length of water way L = P = $4.75\sqrt{Q}$ = 537.401 m ≈537.5m

Intensity of discharge q = Q/L = 23.81 cumecs/m

Assuming sharp crested weir, $q = 1.84\ (H)^{3/2}$

$H = 5.51$ m

Now assume afflux = 1m

D/S HFL = HFL before construction = 205.7 m

U/S HFL = D/S HFL + afflux = 206.7 m

Lacey's silt factor f = 1 (given)

Maximum scour depth, $R = 1.35 \left(\frac{q^2}{f}\right)^{\frac{1}{3}}$ =11.173 m

Approach velocity, $V_a = q/R$ = 23.81/11.173 = 2.131m/sec

Velocity head, $V_a^2/2g$ = 0.231m/sec

U/S TEL = U/S HFL + $V_a^2/2g$ = 206.931m

Now crest level of weir = U/S TEL – H = 206.931 – 5.51 = 201.421m

Under sluices Portion

Design discharge, Q = 3200m^3/sec

Crest level of under sluices = bed level of river = 200.0m

U/S TEL = 206.931m

Total head on the under sluice crest = 206.931 – 200.0 = 6.931m

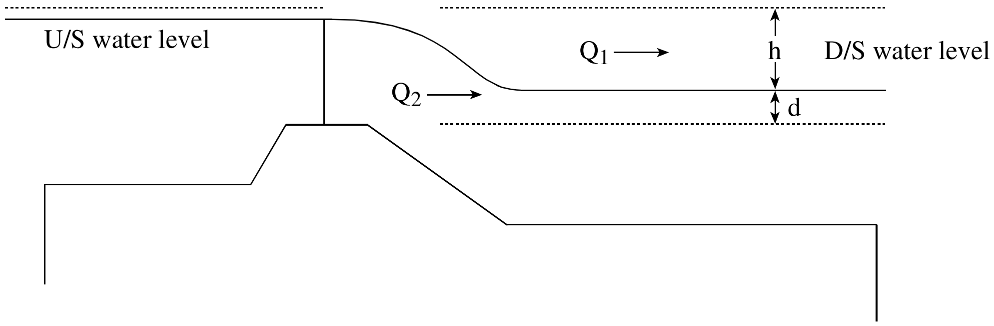
Assuming broad crested weir, $q = 1.71\ (6.931)^{3/2}$ = 31.202 cumecs/m

Length of water way at under sluice = Q/q = 102.55 m

Canal head regulator Portion

Crest level of canal head regulator = crest level of under sluices + 2 = 200 + 2 (assume) = 202m which should be less than full supply level of canal.

Now pond level = full supply level + loss in canal head regulator = 203 + 0.5 (say) = 203.5m



From equation 5.10

$$Q = \frac{2}{3} Cd_1 \times B \times \sqrt{2g} \times [(h + h_a)^{3/2} - h_a^{3/2}] + Cd_2 \times B \times d \times \sqrt{2g \times (h+h_a)}$$

Where $Cd_1 = 0.577$ and $Cd_2 = 0.80$ and ignoring velocity head,

$$Q = B\sqrt{h}\,(1.69h + 3.54d)$$

Here, h_L = pond level - FSL of canal = 203.5 – 203.0 = 0.5m

d = D/S FSL – Crest level = 203.0 – 202.0 = 1m

Now, $200 = B\sqrt{0.5}\,(1.69 \times 0.5 + 3.54 \times 1)$

Now water way for canal head regulator = 64.50m

Exercise 5

1. Explain site selection of Headwork.
2. Differentiate between weir and barrage with sketches.
3. Draw neat sketch of the general layout of a diversion head works and detail explanation of each component of head works.
4. With the help of neat sketches, explain how silt excluders and silt ejectors control the bed load in an irrigation system.
5. Write with definition sketches, how do you determine the pressure along the foundation of the structure and ensure safety against uplift pressure using Bligh's creep theory.
6. Draw four simple Khosla's profiles for a weir of complex profiles. What corrections Khosla suggested to accommodate such simplifications?
7. How the bed load is controlled at head works?

✱✱✱

CHAPTER 6

River Training Works

The expression 'river training' includes all measures adopted on a river to train its flow and regulate the river bed or to increase the low water depth. The purpose of river training is to establish the channel along a certain alignment.

River training is an essential part of any canal development, when water is diverted from a river, after constructing a weir across the river and a head regulator for the channel. Sometimes the alignment of channel can come close to the river or some other stream, and it becomes necessary to divert the menacing river away from the channel (Varshney, 2001).

The rivers in alluvial plains frequently change their courses. They also overflow their bank and cause floods. River training works are required to stabilize the river channel along a certain alignment and with a certain cross section so that river does not cause damage to the land and property adjacent to its bank. River training works are also required at or near various hydraulic structures such as weirs, aqueducts, bridges etc. to prevent outflanking of these structures and minimizing cross flows. In case of weirs, these are also required to provide a favorable curvature of flow at the head regulator for the control of silt entry into the main canal. The main objectives of the river training works are summarized as follows.

- To provide a safe passage to pass flood without over flowing its banks and thus to prevent flooding of the adjoining areas.
- To prevent the river from changing its course and eroding adjoining lands.
- To prevent the erosion of banks and hence improve the alignment by stabilizing the river channel.
- To help the river transport the bed load and suspended sediment load efficiently.
- To prevent outflanking of the hydraulic structures and to guide the river to the structures in a straight and non – tortuous alignment.
- To provide the minimum desired depth of flow required for navigation.
- To provide a favorable curvature of flow at the weir to control entry of silt in to the canal.
- To reduce cross currents in the river which may endanger the hydraulic structures or protection works.
- To deflect the river flow away from a bank or to attract the river flow towards a bank.
- To control and regulate the river bed configuration.

6.1 River Stages and River Characteristics

River morphology is concerned with channel configuration and geometry, and with longitudinal profile; it is time dependent and varies particularly with discharge, sediment input and characteristics, and with bank material. River morphology can be substantially influenced by engineering works, although this influence is not necessarily beneficial. Natural river channels are straight (usually only very short reaches), meandering, i.e. consisting of a series of bends of alternate curvature connected by short, straight reaches (crossings), or braided, i.e. the river divides into several channels which continuously join and separate. The various stages of a schematized river (de Vries, 1985) are shown in Figure 6.1:

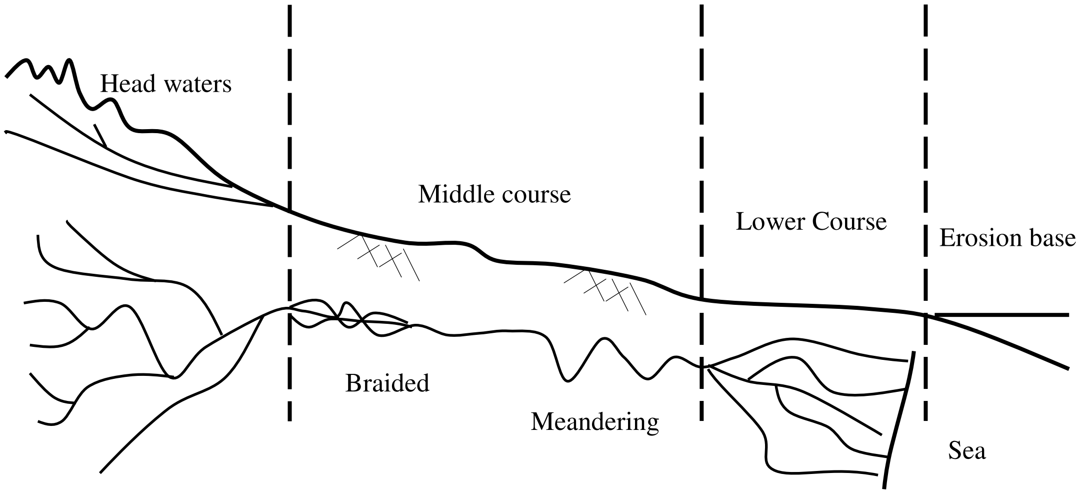


Figure 6.1: River stages and characteristics

River regions stages can be divided according to the topography of river basins. Usually they are four:

- The mountainous - Flashy Rivers
- The sub-mountainous - Incised and Boulder Rivers
- The flood plain - Alluvial Rivers
- The deltaic - Tidal and Deltaic Rivers

The characteristics of different (regions) are shown in Table 6.1 (Varshney, 2001):

Table 6.1 Characteristics of River

River characteristics	Units	Mountainous	Sub – Mountainous	Trough or Alluvial	Deltaic
Bed slope	m/km	115 to 4	4 to 1	1 to 0.1	0.1 to 0.05
Velocity	m/sec	15 to 5	5 to 1.5	1.5 to 0.8	0.8 to 0.3
Course	-	Narrow, fixed & Deep	Well defined	Meandering with in valley to wide meandering	Very wide shallow & changing
Material	-	Heavy stones	Boulders and shingles	Gravel and sand	Silt and sand

6.2 Classification of River Training Works

Depending upon the purpose required, the river training works may be classified into the following three types.

1. **High Water Training Works**

   High water training works are constructed for the purpose of quick disposal maximum flood and to protect adjoining land against damage due to floods. High- water training works are mainly the marginal banks (levees) constructed to protect the adjoining land. Marginal banks of suitable alignment and height are designed to provide sufficient waterway for the safe passage of maximum discharge. High – water training may also include various measures of channel improvement to increase the waterway high – water training is also known as the *training for discharge*.

2. **Low Water Training Works**

   Low water training works are constructed to provide sufficient water depth for navigation during low flows period. Generally, groynes (or spurs) are constructed to contract the width of the channel and hence to increase the depth. Bandalling is also used for closing the side channel and concentrating the flow in the main channel. Low water training is also called the *training for depth.*

3. **Medium –Water Training Works**

   Mean water training works are constructed for efficient disposal of the bed sediment load and suspended sediment load of the river so that it remains in good shape and condition. The maximum accretion capacity of a river occurs when the flow is at the mean value and the discharge is equal to the dominant discharge. Depending upon the stage of flow, the required changes in the river bed are made. Mean water training is also known as the *training for sediment*.

   The mean – water training is the most important training of the three types. The maximum movement of sediment occurs during the mean water stage. A river – training work constructed to change the river cross – section and alignment should be designed in accordance with this stage. During a high flood stage, the activity f the bed material of the river is maximum but it lasts for a very short duration. On the other hand, the activity of the bed material of the river is minimum at low stages which persist for a long duration. In between these two extreme stages, there is the mean flow stage at which combining effects of forces causing sediment movement and the duration for which these forces are maintained is a maximum. .during the mead – water stage, there is considerable influence on the configuration of the river. These stage, therefore, controls the design of most of the river training works. In fact, mean water training forms the basis on which both high and low water training are also planned.

6.3 Methods of River Training Works

The following are the various methods of river training and control works in vogue:

1. Marginal banks (or levees)
2. Guide banks (or Bell's bunds)
3. Groynes (or spurs)
4. Artificial cut-off
5. Bank protection and pitched banks,
6. Pitched islands
7. Sills (or submerged dikes)
8. Bandalling
9. Closing dikes

1. Marginal Embankments

Marginal embankments (or levees or dikes or marginal banks) are the earthen embankments which are constructed parallel to the river banks to confine the flood waters within the cross – section available between them. These are usually placed at some distance away from the river bank so that the area of flow is increased Figure 6.2. Generally, the marginal embankments are on both sides of the river channel, but if the topography is such that one bank is already quite high, there would be a marginal embankment only on the other side. The marginal embankments prevent the spread of flood water over the adjoining land. The purpose of these embankments is mainly to control the river flood. The marginal embankments are generally required for the meandering rivers. The alignment of these embankments follows the normal pattern of meandering of the river. The distance by which an embankment is constructed away for the main channel. Is called retirement. Besides the technical factors, human and political factors should also be considered while deciding the retirement because the land laying between the two marginal embankments on the opposite banks remains unprotected from floods.

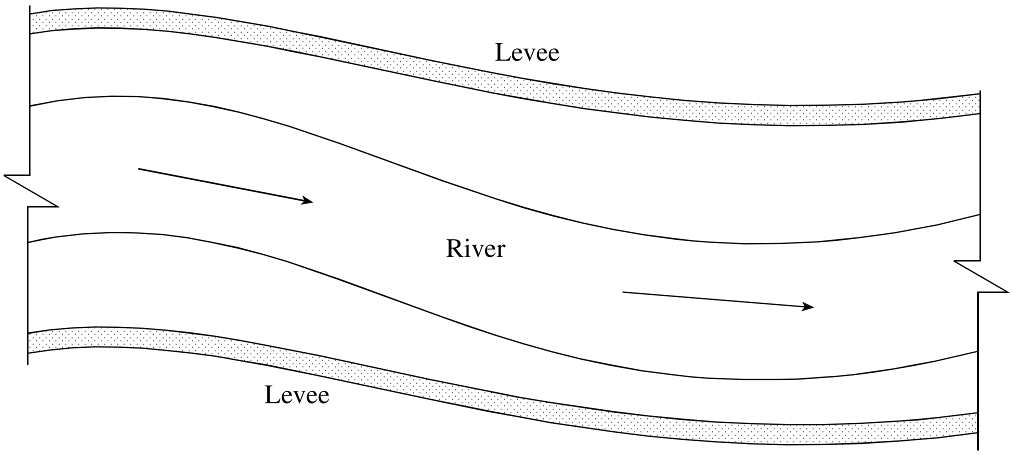


Figure 6.2: Marginal Embankments

2. Guide Banks:

While selecting a site for hydraulic structure on river crossing like barrage, weir, cross drainage structures bridges, culverts on an alluvial river certain requirements are always kept in mind. These requirements include straight reach of the river and small width of the river at the bridge site. The crossing reach between two successive bends of a meandering site is suitable from these considerations. However, the meandering pattern itself migrates and, hence, steps must be taken to ensure that the flow path does not change through the waterway at the bridge site, and also that the approach road embankment is not endangered due to the smaller waterway provided. For this purpose, earthen embankments are provided on one or both sides of the river at the bridge site. These embankments are known as guide banks (or guide bunds).

The layout of the guide banks should be such as to guide the flood smoothly through a work constructed across the river. The guide banks are provided in pairs symmetrical in plan and may either be kept parallel or converge slightly towards the work, and usually extend a little distance downstream from the abutments of the work. They are meant to confine the river within a reasonable waterway, and direct the flow in a manner that would ensure its safe and expeditious passage. They should also protect the works from outflanking, and adjacent land from flooding due to the afflux caused as a result of construction of an obstruction in the river.

The effect of confining the river between the guide banks is to:

a. Increase the rate at which the flood wave travels down the stream

b. Increase the water surface elevation of the river at flood

c. Increase the maximum discharge at all points downstream

d. Increase the velocity and the scouring action through the leveed section

e. Reduce the surface slope of the stream above the leveed portions.

The method of constriction a river for the construction of a bridge and controlling is to make is flow axially through the bridge and controlling is to make is flow axially through the bridge by means of a system of guide bunds (first introduced by Bell). This system known as Bell's guide bunds system, has become a standard practice.

3. Groynes or Spurs

Groynes are structures constructed transverse to the river flow and extend from the bank into river. These structures are known by several names, the most popular being spurs, Cut-off spur dikes and transverse dikes, they are probably, the most widely used training works.

(i) Functions of Groynes

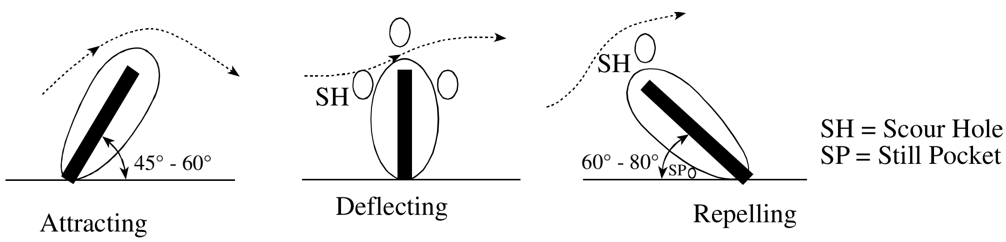
Groynes serve one or more of the following functions:-

- Training the river alone a desired course by attracting, deflecting or repelling, the flow in a channel;
- Creating a slack flow with the object of silting up the area in the vicinity;
- Protecting the river bank by keeping flow away from it; and
- Contracting a wide river channel, usually for the improvement of depth for navigation.

(ii) Types of Groynes

Groynes can be classified as follows:-

- Classification according to method and materials of construction – permeable and impermeable (solid)
- Classification according to the height of spur below high water – submerged or non-submerged
- Classification according to the function served – attracting, deflecting and repelling.
- Special types - Denehy's T-headed groynes, Hockey type, Burma Type etc.



(iii) Methods of use

Factors which influence the choice and design of groynes are:

- Fall and velocity of river
- Character of bed material carried, such as shingle, boulders, sand or silt.
- Width of waterway, height of flood rise and nature of flood hydrograph
- Available materials and funds.

(iv) Classification according to methods and materials of construction

According to methods and materials of construction the groynes may either be permeable or impermeable.

Permeable Spurs:

Permeable spurs permit significant seepage through them. These groynes simply obstruct the flow and reduce the velocity of flow to cause deposition of silt. These are generally used for rivers carrying a huge sediment load in suspension. As the sediment accumulates between the groynes, it protects them and there is no necessity of any other material for its protection. When permeable groynes are constructed in rivers having clear water with little silt load, they reduce the erosive power of the river current and thus prevent bank erosion. The permeable groynes are temporary structure and are susceptible to damage by floating debris. Because a permeable groynes does not abruptly change the direction of flow, intense eddies and severe scour holes generally do not occurs because of its construction.

Impermeable Spurs:

Impermeable groynes are earthen (or rock filled) embankments duly protected on the sides and at top by a strong covering of stone pitching or concrete blocks. Impermeable groynes do not permit significant flow of water through them. The impermeable groynes are also known as solid groynes or embankment groynes.

The design of impermeable groynes is similar to that of guide banks. The side slopes generally vary from 2:1 to 3:1, depending upon the material used. However, the slopes at the front end (or head or nose) of the groynes is usually between 3:1 and 5:1. Because the head of the groynes is subjected to severe attack by the river current, a launching apron must be provided on it. The thickness of pitching is also increased at the head. However, the impermeable groynes is pointed towards downstream for attracting the flow or pointed towards upstream for repelling the flow. When it is perpendicular to the bank, it deflects the flow.

4. Artificial Cutoff

An artificial cutoff is developed in a meandering river to divert the flow along a straight course. .an artificial cutoff is used for the purpose of training of river when its meander goes an increasing and endangers some valuable land or property on its banks. It is also used for straightening the river approach to a structure. It has been observed in practice that a natural cutoff in a meandering river occurs when the cutoff ratio (i.e. arc/ chord ratio) exceeds 1.7 to 3.0. An artificial cutoff can be induced when the cutoff ratio is in this range. To induce an artificial cutoff, a small pilot channel to carry 8 % to 10% of the river discharge is excavated along the proposed line of the artificial cutoff. The flood water gradually enlarges the pilot channel to the required section when the river flows in an erodible material. Ultimately, the river takes the course, where R is the hydraulic radius and L is the length. A deep and hydraulically efficient pilot channel is required for inducing the artificial cut. Because the tractive force is directly proportional to the depth of the channel, a deep cut is useful for the rapid development of the cutoff.

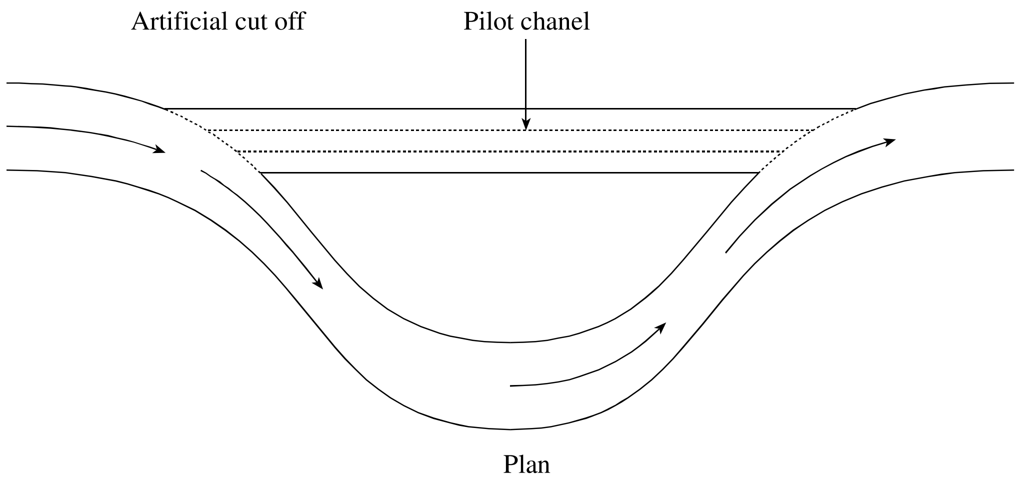


Figure 6.3: Artificial cutoff

5. **Bank Protection and Pitched Banks**

Purpose of the bank protection may be anyone or more of the following:

- Training the river
- Protection of adjacent land and valuable property
- Protection of hydraulic structures
- Protection of flood embankment
- Affording facilities for water transportation

Bank protection works may be classified as direct and indirect. Direct protection includes works done on the bund itself such as providing vegetation cover, pavement, pitching and grading of slope etc. Indirect protection includes works constructed not directly on the banks, but in front of them for deducing the erosive action of the current. If the current is strong protection has to be provided by stone pitching or various types of mattress such as willow, asphalt or articulated concrete. Stone is the most commonly used material for protection of banks where available locally. The thickness of pitching required is governed by the velocity of the current near the bank.

6. **Pitched islands**

A pitched island is an artificially created island in the river bed, either made of masonry or stone pitched on all sides. On account of the turbulence created by the island placed in a series, a deep river channel is created and flow is diverted away from the bank in danger. These are used for the following purposes:

- To train the river to have an axial flow
- To improve channel for navigation purpose
- To correct the adverse curvature for effective sand exclusion

7. Bandalling

Bandalling is a method of river training which is used to confine the low water flow of a shallow river in a single channel for maintaining the required depth for navigation. Bandalling is started after the flood season is over and the water level begins to fall. A bandal consists of a framework of vertical bamboos, 3 to 6m long, driven into the river bed. The bamboos are driven in a line at a spacing of 0.6m and are held in position by means of horizontal ties and are supported by struts at every 1.2m. A number of such bandals are erected depending upon the width of river. Bamboo mattings are tied to bamboo frameworks with the help of coir ropes. These mattings are usually 0.9m wide and are strengthened at the edges by strips of split bamboos. Bandals are usually erected at an angle of 30^0 to 40^0 to the direction of flow, inclined downstream.

Check the flow and cause sediment deposition parallel to and behind them. Thus a deep channel is formed in front of the bandals with sand deposition on either side. The whole discharge of the river is now directed through this deep channel, and therefore the depth of flow is increased. The process of deepening of the channel usually take about 2 to 3 weeks. Once a channel has deepened, it remains in that condition till the next flood. It does not require much maintenance. After the flood has receded, bandals are again constructed.

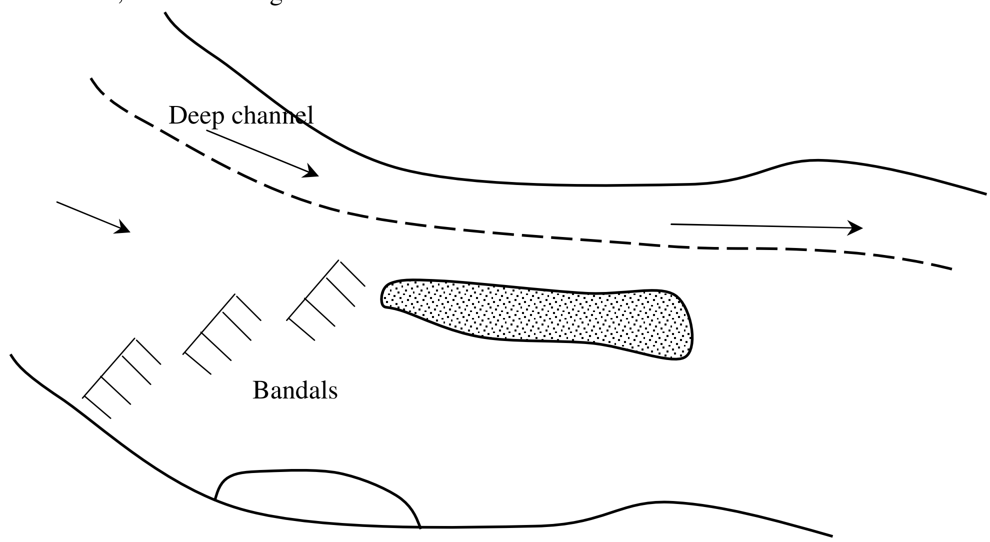


Figure 6.4: Bandalling

6.4 Design of Guide Bund and Launching Apron

Guide banks are artificial embankments meant for guiding the river flow past a bridge (or other hydraulic structures such as weirs or barrages) without causing damage to the bridge and its approaches. Guide banks are built along the flow direction both upstream and downstream of the structure on one or both sides of the river as desired. Guide banks for a bridge restrict the waterway at the bridge site and prevent the outflanking of the bridge by the changing course of the river. The design criteria of guide banks are based on the works of Spring and Gales.

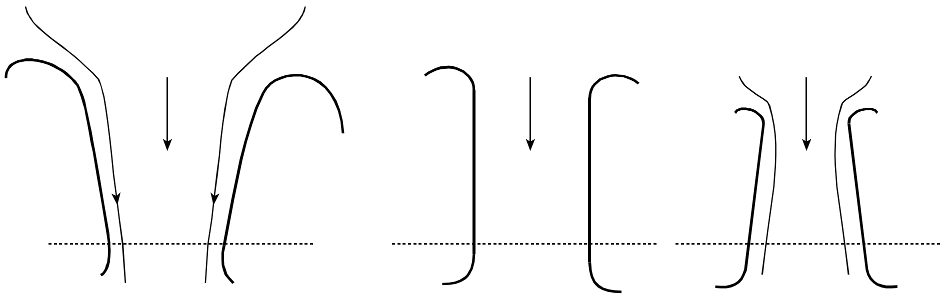
1. Layout of Guide bund

In general the guide bunds may be classified as

(a) Divergent (b) Convergent (c) Parallel upstream of the work

Divergent bunds exercise an attracting influence on the flow. They are suitable where the river current has been oblique to the structure. Such bunds have the disadvantages of shoal formation in between them, if splay is excessive.

Convergent bunds have the disadvantage of excessive attack and heavy scour at the head and shoaling all along the bank, rendering the end bays ineffective. Due to these drawbacks convergent guide bunds are rarely used. Thus symmetrical and parallel guide bunds are generally used unless the local conditions dictate otherwise. The actual layout of the guide bunds depends upon the local topography site of the structure, alignment of approach embankment, afflux bunds and off-taking channels.



(a) Divergent (b) Parallel (c) Convergent

Figure 6.5: Layout of Guide bund

2. Length of water way

The design of a hydraulic structure on an alluvial river is the estimation of the minimum and also a safe waterway. A reasonable estimation of clear waterway to be provided between guide banks can be obtained by equating it to Lacey's regime perimeter given by following equation.

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

Where, P = Lacey's Regime perimeter in meter

Q = Design flood discharge in cumecs.

The width of large river is approximately equal to its wetted perimeter in alluvial plains. In case of bridge, weir and barrage, obstruction caused by piers should be accounted for, and the above equation should be taken to represent the clear effective waterway. It should be remembered that the regime conditions are disturbed after the construction of the weir, and the above formula is not strictly applicable. Most of the existing weirs and bridges have been provided with a clear waterway from 10% to 50% more than that given by Lacey's Regime Perimeter. Mathematically, Length of waterway L = (1.1 to 1.5)P

3. Length of Bund

Spring and Gales co-related the length of the guide bund with the length of the structure (water weir) between abutments (L) and have recommended the upstream length of the guide bunds as 1.25L for flood discharge up to 20,000 cumecs and 1.5L for flood discharge more than 20,000 cumecs.

In some cases where the channel bank is wide, the guide bunds according to this criterion may not provide enough protection to the approach embankments. Therefore in such cases the length and the layout of the guide bunds should be determined in a manner that the approach embankments are safe against the worst possible embankment.

If the distance of the channel edge is large, a double loop may be formed. The alignment of the approach embankment plays an important role in deciding the length of the guide bund. The approach embankment curved in the upstream are protected in the zone of possible meander and are frequently attacked in spite of the provision of suitable guide bunds. The approach embankment should, therefore be located along the axis of work up to the channel edge preferably with a curve towards downstream to counterbalance the inadequacy of the guide bunds.

4. Radius of curved head of the guide banks

(a) Upstream curved portion

The upstream curved portion of a guide bank is called the impregnable head. The radius of curvature of the impregnable head should be sufficient enough, as not to cause intense eddies due to the curved flow near it. Greater the radius, flatter the curve, and lesser is the probability of formation of eddies. For a same river slope, coarser the bed material, shorter can be the radius depending on the expected velocity. A safe value for the radius R may be taken equal to $R= 0.45\ L$

Where, R= radius of the curved head and

P= length of water way

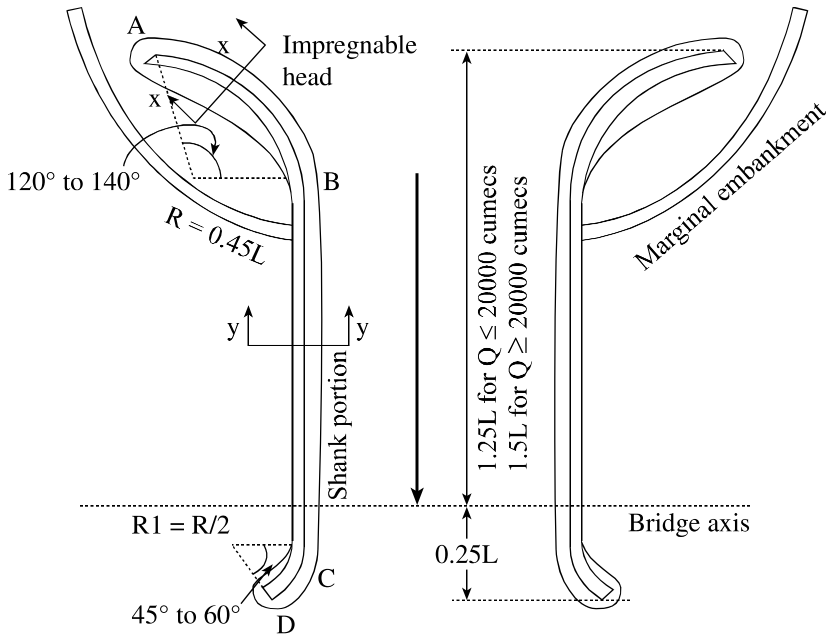
However, Spring suggested a value of R equal to 180 to 250 m having velocities between 2.4 to 3.4 m/sec respectively.

Gales, on the other hand, suggested a value of R= 250m for rivers having high flood discharge between 7,000 to 20,000 cumecs, and a value of R= 580 m for discharges 40,000 to 70,000 cumecs. For intermediate discharge i.e. between 20,000 to 40,000 cumecs, the value of R may be obtained by interpolation.

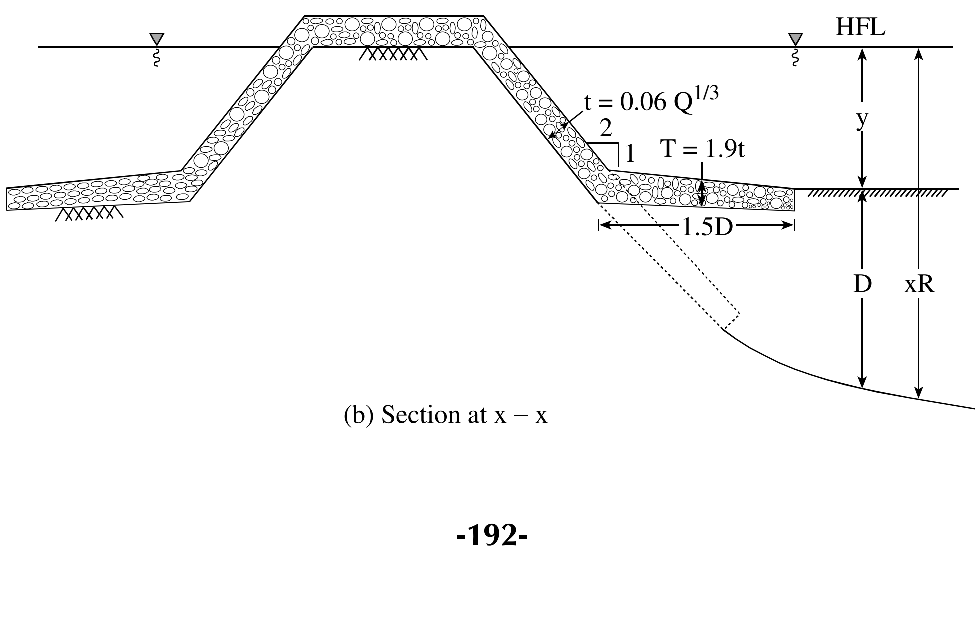
The upstream curve is extended to subtend an angle of 120° to 140° at its center as shown in Figure 6.6.

(b) Downstream curved portion

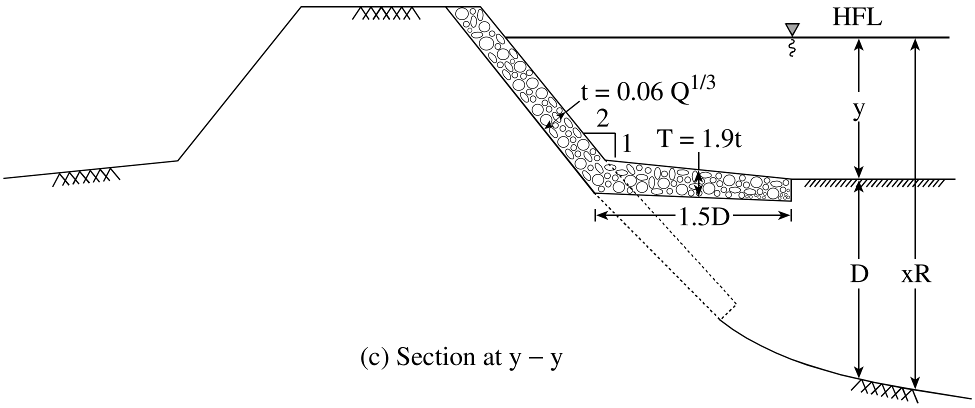
On the downstream the river fans outs as to attain its normal width. The downstream portion of the guide bank ensures the safety of approach embankments and prevents the river from attacking them. This purpose can be well served by providing short guide bund with sharp curved head. A radius equal to half of the radius at the upstream side may be adopted here and sweep angle of 45° to 60° may be provided.



(a)



(b) Section at x – x



(c) Section at y – y

Figure 6.6: Gale's recommendations for guide banks

5. Shank portion and selection of bund

The straight portion of the guide bank BC (in Figure 6.6(a)) is called the shank portion. The guide banks should have a minimum top width of 4 meters so as to provide sufficient carriage way. Extra width may, however be provided for storing pitching materials, etc. Side slopes may vary from 1.5H :1V to 2.5H:1V. (But generally kept as 2H:1V), depending on the construction materials and the height of the bund. A freeboard of 1.2 to 1.5 m is generally provided.

6. Slope Pitching

The river side should be pitched with one man stone weighing 40 to 50 kg or concrete blocks. The rear shank slope may be covered with a coating of 0.3 to 0.6 m earth for growth of vegetation to resist rain and wind erosion.

The thickness of pitching on the river side is given by the relation

$t = 0.06\ Q^{1/3}$ [Where Q= discharge in cumecs]

t = Thickness of stone pitching in m.

7. Launching apron

The face of the guide bund is protected at river bed level by stone pitching so that during floods the sloping face is not damaged. Scour would occur at the toe with consequent undermining and collapse of the stone pitching. To obviate such damage to the slopes, a stone cover known as apron is laid beyond the toe on horizontal river bed so that scour undermines the apron first starting at its farther end and work backwards towards the slope. The apron then launches to cover the face of the scour, with stones forming a continuous carpet below the permanent slopes of the guide bunds. Adequate quantity of stones for apron has to be provided to ensure complete protection of the whole of the scoured face. Generally scour slope of 2H:1V is assumed. In this case the length of the scoured face will be equal to $\sqrt{5}\ D$. The thickness of apron in launched position is assumed as 1.25t, where t is the thickness of slope pitching. Then the volume of stone required in the launched apron per unit length= $\sqrt{2^2 + 1^2} \times D \times 1.25t = 2.5\ t\ D$.

The launching apron is generally laid in a width equal to 1.5 D. where D is scour depth below the original bed. The total scour below HFL is taken as xR, where R is the Lacey's normal scour depth given by equation. $R=0.47\left(\frac{Q}{f}\right)^{\frac{1}{3}}$

Where Q is the discharge and f is the silt factor, values of x are tabulated for different below places of guide band.

S. N.	Locations	Mean value of x	D = xR – y (water depth above bed)
1.	Noses of guide banks	2.25	2.25R – y
2.	Transitions from noses to straight portions	1.5	1.5R – y
3.	Straight reaches of guide bunds	1.25	1.25R – y

If the width of unlaunched apron is 1.5D, then the thickness of the unlaunched apron T is given as:

$$T = \frac{2.8 \times t \times D}{1.5D} = 1.87t = 1.9t$$

Example 6.1:

A bridge is to be constructed across a river, having the following hydraulic data:

- Maximum discharge = 17000 cumecs
- Highest flood level = 288.0 m
- River bed level = 280.0 m
- Average diameter of the river bed material = 0.10 mm

Find out the desired waterway for the bridge. Design Bell's guide bunds including launching apron and also guide banks on either side to train the river. Big boulders are available in a hilly torrent nearby.

Solution:

Step 1: Length of waterway

Lacey's regime perimeter

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

$$P = 4.75\sqrt{17000} = 619.32 \text{ m}$$

Allowing 20% extra for piers etc. the length of the overall waterway between two guide banks at the bridge site is obtained as:

$$L = 1.2 \times 619.32 = 743.189 \approx 744 \text{ m}$$

Step 2: Length of guide bank

The length of the guide bank upstream of bridge = 1.25 × L = 1.25 × 744 = 930 m

The length of the guide bank downstream of bridge = 0.25 × L= 0.25 × 744= 186 m

Step 3: Radius of Curve head

Following the recommendations of Gales,

The radius of U/S curved head of the guide bank = 0.45 × L = 334.8 m

The U/S curved head may be provided with a sweep angle of 135^{O}.

The radius of D/S curved head of the guide bank = R/2 = 334.8/2 = 167.4m

and may be provided with a sweep angle of 60^{O}.

Step 4: Cross section of Guide bank

The top width of the guide bank is assumed as 4 m and the side slopes as 2:1

H.F.L. at the bridge site = 288.0 m

Assume free board = 1.5m

Assume nil value of afflux and neglect velocity head.

Top level of guide bank = 288.0 + 1.5 = 289.5 m

River Bed level = 280.0 m

Now, Height of guide bank above the bed of the river = 289.5 – 280.0 = 9.50m

Step 5: Slope protection

Thickness of stone pitching on the side slopes of the guide bank is given by

$$t = 0.06\ Q^{1/3} = 0.06 \times 17000^{1/3} = 1.54\text{m} \approx 1.55\ \text{m}$$

The volume of stone required for pitching per meter length = $\sqrt{5} \times 9.50 \times 1.55 = 32.926\ m^3/m \approx 33\ m^3/m$

Step 6: Launching Apron

Lacey's regime scour depth is given by

$$R = 0.47\left(\frac{Q}{f}\right)^{\frac{1}{3}} = 14.66\ \text{m}$$

$$f = 1.76\sqrt{d_{mm}} = 0.56$$

Depth of water above river bed (y) = 8m

At the nose:

Depth of scour below the river bed (D) = xR – Y = 2.25 × 14.66 – 8 = 25m

Length of launching apron = 1.5 D = 1.5 × 25 = 37.5 m

Thickness of launching apron = 1.9 t = 1.9 × 1.55 = 2.945m

At transition of Guide bank:

Depth of scour below the river bed D = xR – y = 1.5 × 14.66 – 8 = 14m

Length of launching apron = 1.5D = 21m

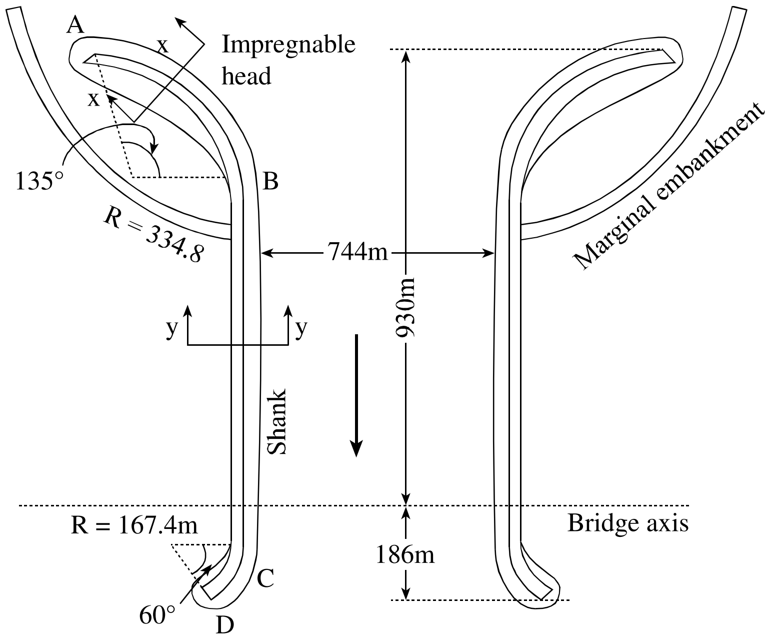
Thickness of launching apron = 1.9 t = 1.9 × 1.55 = 2.945m

At the shank of Guide bank

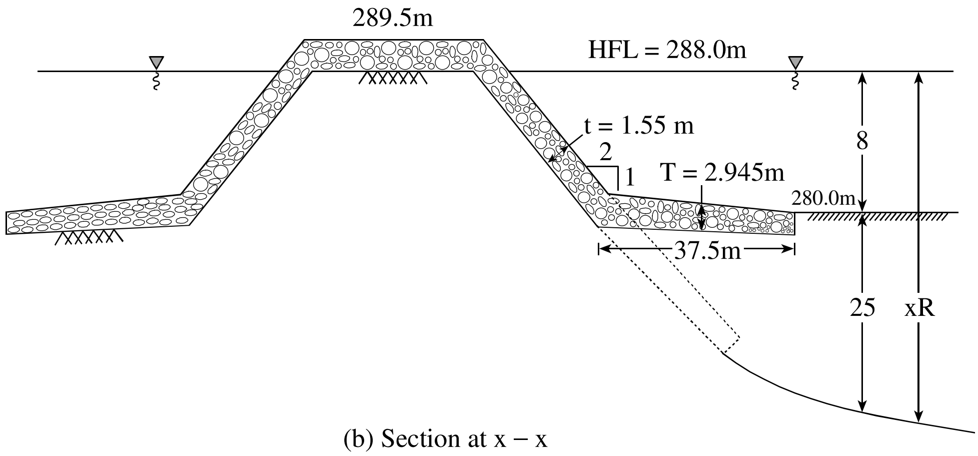
Depth of scour below the river bed D = xR – y = 1.25 × 14.66 – 8 = 10.5m

Length of launching apron = 1.5D = 15.75m ≈ 16m

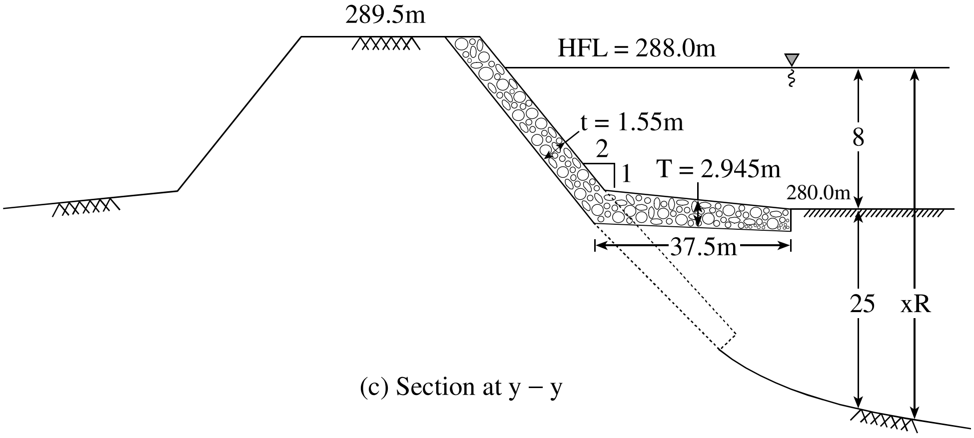
Thickness of launching apron = 1.9 t = 1.9 × 1.55= 2.945m



(a)



(b) Section at x – x



(c) Section at y − y

6.5 Design of Spur (Groynes)

Spurs (or jetties, as they are often called) are defined as linear structures, permeable or impermeable, projecting into a channel from the bank for the purpose of altering flow direction, channel bank protection, inducing deposition, or reducing flow velocity along the bank. Design of spur consist of permeability, length and layout of spur and size/ height of spur.

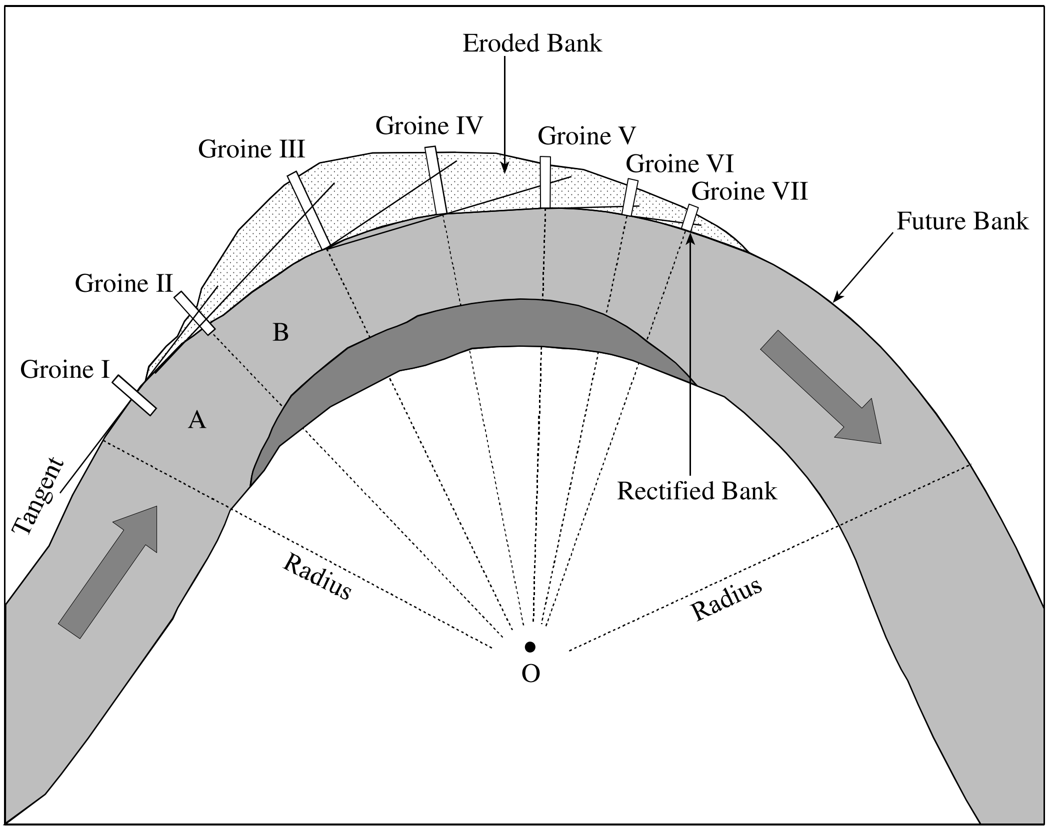


Figure 6.7: Layout of spur in river bend

Permeability

Where it is necessary to provide a significant reduction in flow velocity, a high level of flow control, or where the structure is being used on a sharp bend, the spur's permeability should not exceed 35 percent.

- Where it is necessary to provide a moderate reduction in flow velocity , a moderate level of flow control, or where the structure is being used on a mild to moderate channel bend, the spurs with permeability up to 50 percent can be used.
- In environments where only a mild reduction in velocity is required, where bank stabilization without a significant amount of flow control is necessary, or on mildly curving to straight channel reaches, spurs having effective permeability up to 80 percent can be used. However, these high degrees of permeability are not recommended unless experience has shown them to be effective in a particular environment.
- It is recommended that jack and tetrahedron retardance spurs be used only where it can be reasonably assumed that the structures will trap a sufficient volume of floating debris to produce an effective permeability of 60 percent or less.
- It is recommended that Henson-type spurs be designed to have an effective permeability of approximately 50 percent.
- The greater the spur permeability, the less severe the scour pattern downstream of the spur tip. As spur permeability increases, the magnitude of scour downstream of the spur decreases slightly in size, but more significantly in depth.
- The vertical structural members of permeable spurs should be round or streamlined to minimize local scour effects. The greater the spur permeability, the lower the magnitude of flow concentration at the spur tip.
- If minimizing the magnitude of flow deflection and flow concentration at the spur tip is important to a particular spur design, a spur with permeability greater than 35 percent should be used.
- The more permeable the spur, the shorter the length of channel bank protected downstream of the spur’s riverward tip.
- Spurs with permeability up to approximately 35 percent protect almost the same length of channel bank as do impermeable spurs; spurs having permeability greater than 35 percent protect shorter lengths of channel bank, and this length decreases with increasing spur permeability.
- Because of the increased potential for erosion of the channel bank in the vicinity of the spur root and immediately downstream when the flow stage exceeds the crest of impermeable spurs, it is recommended that impermeable spurs not be used along channel banks composed of highly erodible material unless measures are taken to protect the channel bank in this area.

Extent of Channel bank Protection

A common mistake in stream bank protection is to provide protection too far upstream and not far enough downstream.

- The extent of bank protection should be evaluated using a variety of techniques, including:
  - Empirical methods,
  - Field reconnaissance,
  - Evaluation of flow traces for various flow stage conditions, and
  - Review of flow and erosion forces for various flow stage conditions.

Information from these approaches should then be combined with personal judgment and a knowledge of the flow processes occurring at the local site to establish the appropriate limits of protection.

Spur Length

- As the spur length is increased,
  - The scour depth at the spur tip increases,
  - The magnitude of flow concentration at the spur tip increases,
  - The severity of flow deflection increases, and
  - The length of channel bank protected increases.
- The projected length of impermeable spurs should be held to less than 15 percent of the channel width at bank-full stage.
- The projected length of permeable spurs should be held to less than 25 percent of the channel width. However, this criterion depends on the magnitude of the spur's permeability. Spurs having permeability less than 35 percent should be limited to projected lengths not to exceed 15 percent of the channel's flow width. Spurs having permeability of 80 percent can have projected lengths up to 25 percent of the channel's bank-full flow width. Between these two limits, a linear relationship between the spur permeability and spur length should be used.

Spur Spacing

- The spacing of spurs in a bank-protection scheme is a function of the spur's length, angle, and permeability, as well as the channel bend's degree of curvature.
- The direction and orientation of the channel's flow thalweg plays a major role in determining an acceptable spacing between individual spurs in a bank-stabilization scheme.
- Reducing the spacing between individual spurs below the minimum required to prevent bank erosion between the spurs results in a reduction of the magnitude of flow concentration and local scour at the spur tip.
- Reducing the spacing between spurs in a bank-stabilization scheme causes the flow thalweg to stabilize further away from the concave bank towards the center of the channel.
- A spacing criteria based on the projection of a tangent to the flow thalweg, projected off the spur tip should be used.

Spur Angle/Orientation

The primary criterion for establishing an appropriate spur orientation for the spurs within a given spur scheme is to provide a scheme that efficiently and economically guides the flow through the channel bend, while protecting the channel bank and minimizing the adverse impacts to the channel system.

- Spurs angled downstream produce a less severe constriction of flows than those angled upstream or normal to flow.
- The greater an individual spur's angle in the downstream direction, the smaller the magnitude of flow concentration and local scour at the spur tip. Also, the greater the angle, the less severe the magnitude of flow deflection towards the opposite channel bank.
- Impermeable spurs create a greater change in local scour depth and flow concentration over a given range of spur angles than do permeable spurs. This indicates that impermeable spurs are much more sensitive to these parameters than are permeable spurs.
- Spur orientation does not in itself result in a change in the length of channel bank protected for a spur of given projected length. It is the greater spur length parallel to the channel bank associated with spurs oriented at steeper angles that results in the greater length of channel bank protected.

- Retardance spurs should be designed perpendicular to the primary flow direction.
- Retardance/diverter and diverter spurs should be designed to provide gradual flow training around the bend. This is accomplished by maximizing the flow efficiency within the bend while minimizing any negative impacts on the channel geometry.
- The smaller the spur angle, the greater the magnitude of flow control as represented by a greater shift of the flow thalweg away from the concave (outside) channel bank.
- It is recommended that spurs within a retardance/diverter or diverter spur scheme be set with the upstream-most spur at approximately 150 degrees to the main flow current at the spur tip, and with subsequent spurs having incrementally smaller angles approaching a minimum angle of 90 degrees at the downstream end of the scheme.

Spur System Geometry

- A step-by-step approach to setting out the geometry of a retardance/diverter or diverter spur scheme was presented above. The use of this approach will yield an optimal geometric spur system design.

Spur Height

- The spur height should be sufficient to protect the regions of the channel bank impacted by the erosion processes active at the particular site.
- If the design flow stage is lower than the channel bank height, spurs should be designed to a height no more than three feet lower than the design flow stage.
- If the design flow stage is higher than the channel bank height, spurs should be designed to bank height.
- Permeable spurs should be designed to a height that will permit the passage of heavy debris over the spur crest and not cause structural damage.
- When possible, impermeable spurs should be designed to be submerged by approximately three feet under their worst design flow condition, thus minimizing the impacts of local scour and flow concentration at the spur tip and the magnitude of flow deflection.

Spur Crest Profile

- Permeable spurs should be designed with level crests unless bank height or other special conditions dictate the use of a sloping crest design.
- Impermeable spurs should be designed with a slight fall towards the spur head, thus allowing different amounts of flow constriction with stage (particularly important in narrow-width channels), and the accommodation of changes in meander trace with stage.

Channel bed and Channel bank Contact

- Careful consideration must be given to designing a spur that will maintain contact with the channel bed and channel bank so that it will not be undermined or outflanked. Methods and examples presented herein can be used to ensure adequate bend and bank contact.

Spur Head Form

- A simple straight spur head form is recommended,
- The spur head or tip should be as smooth and rounded as possible. Smooth, well-rounded spur tips help minimize local scour, flow concentration, and flow deflection.

Exercise

1. What is river training works? Explain with sketch three methods of training works normally adopted in Nepalese river.
2. Design the length, radius of curved head, length and thickness of launching apron of a guide bund to train a river with the following data.

   Design flood discharge = 4500 cumecs; Bed level of river = 150.00m

   HFL = 154.00m; average diameter of river bed material = 0.1mm
3. Explain with sketch how spur assist in river control work.
4. The launching apron of a guide bank is laid in a width equal to 1.8 times the depth of scour below original bed. If a scour slope of 3:1 is to be maintained with thickness 1.5t, find the thickness of apron before it get launched. Draw neat sketch of designed structure.
5. Calculate the thickness of a 7m long launching apron of loosed stones for a shank portion of guide bund at a bridge site of a river having design flood of 8000 m3/sec and flood depth of 5m. Assume that average diameter of river bed material at flood time is 0.3mm.

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CHAPTER 7

Regulating Structures

The structures (or masonry works) constructed on a canal to control and to regulate discharge, velocity, depth etc. are known as canal regulation works. These structures are required for proper and efficient functioning of an irrigation canal system. The water which enters the main canal through a head regulator installed at the canal headworks is distributed into different branches and distributaries. To distribute the water effectively, the discharge is regulated in these smaller channels.

The distributary head regulator is constructed at the head of a distributary (or a branch canal) to control and to regulate the flow of water into the distributary. Thus a distributary head regulator provides a necessary link between the parent channel and the offtaking channel.

A cross – regulator is generally constructed across the parent channel at the downstream of the off – take point of the offtaking channel for raising up the water level in the parent channel when its discharge is less than the full supply discharge. Cross regulators are also constructed for various other purposes.

A canal fall is a structure constructed on a channel to effect a sudden change in the bed level of the channel. Canal falls are required on a channel when the slope of natural ground along the alignment of a channel is steeper than the bed slope of the channel. If a fall is not provided, the channel will go in excessive filling, which is not desirable.

The offtaking channel should take its proportionate share of sediments of the parent channel. Various silt-excluding devices are provided at the distributary head regulator. The aim of all such devices is to separate the bottom layers of water charged with the concentrated sediment load from the top layers and to draw top layers of clear water into the offtaking channel without causing any disturbance.

7.1 Alignment of the Off – taking Canals

A channel taking off from another large channel (called parent channel) should be properly aligned with respect to the parent channel. The alignment should be such that the off – taking channel is able to draw its supply without any undesirable effect. The following types of alignment are commonly adopted in practices.

1. The best alignment of the offtaking channel is when it makes nearby zero angle with the parent channel initially and then separates out gradually along a transition curve as shown in Figure 7.1. The transition curve should be properly designed to avoid accumulation of silt in the form of a silt jetty and to ensure equitable distribution of silt.

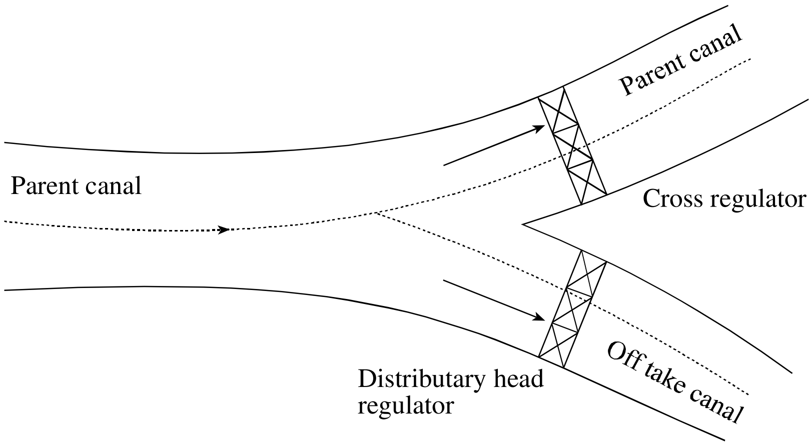


Figure 7.1: CR and DHR

2. If the transition curve are not provided, the alignment shown in Figure 7.2 may be adopted. In this case, the offtaking channel as well as the parent channel on downstream make an angle with the parent channel on upstream of the offtake point.

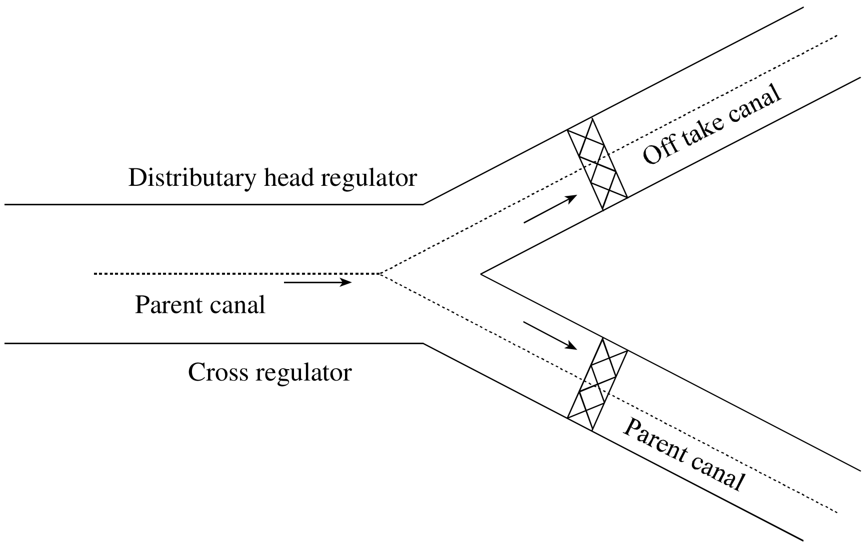
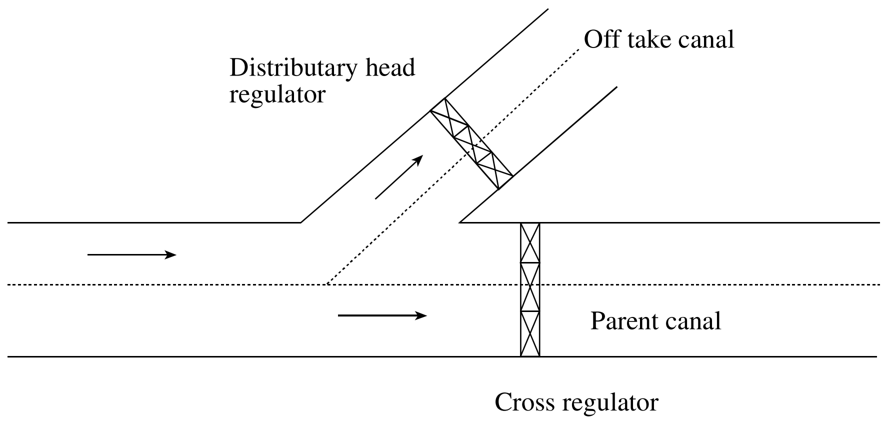


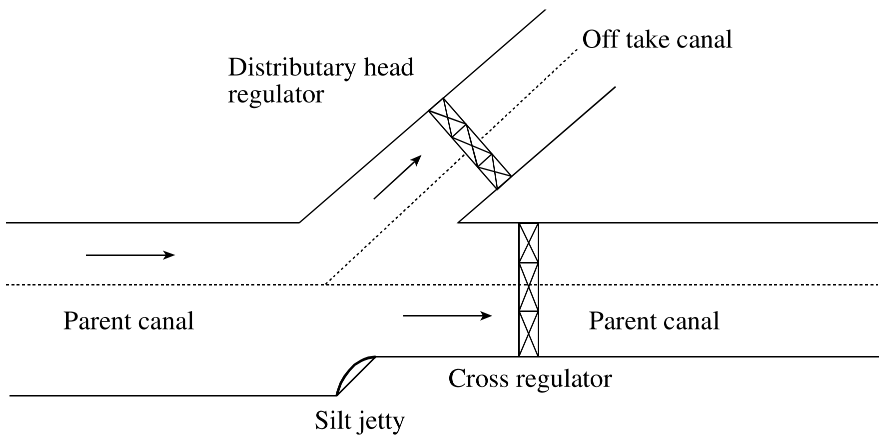
Figure 7.2: CR and DHR

3. If it is essential to keep the parent channel straight the edge of the channel rather than the center line should be considered while deciding the angle of off take as shown in Figure 7.3 (a). An angle of 60^0 to 80^0 is quite suitable. However, for large and important works the model studies should be conducted to determine the most suitable angle.

Figure 7.3 (b) shows an unbalanced offtake, which should be avoided as far as possible. This usually results in the formation of silt jetty. Moreover, the deviated current of water scours the bed along the deviated line to make up the loss of silt due to jetty formation.



(a)



(b)

Figure 7.3: CR and DHR

4. When a number of channels off-take from one parent channel, the alignment shown in Figure 7.4 is generally adopted. In this case, the off-taking channels are generally flumed to accommodate all the off-takes in the normal width of the parent channel. In this case, only one cross – regulator serves the purpose of a number of cross – regulators which would have been required if separate off – take points were provided for different off-taking channels.

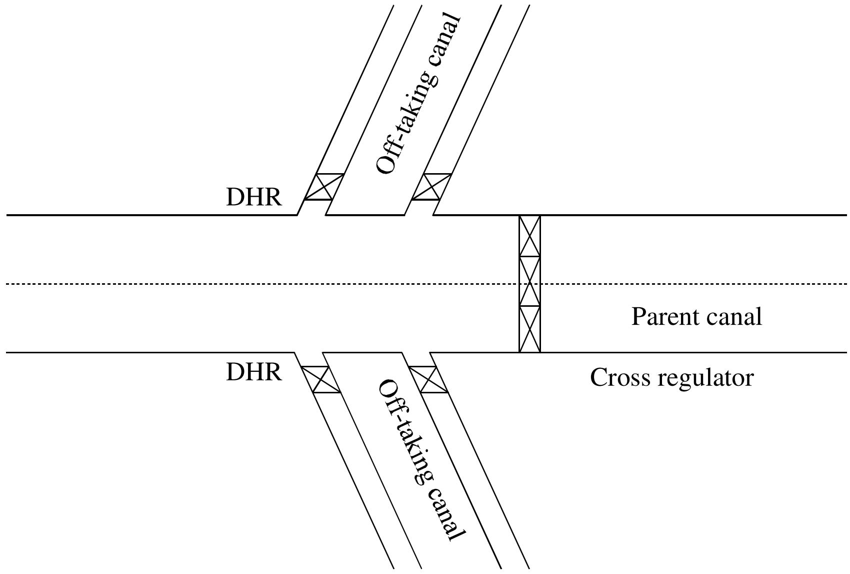


Figure 7.4: CR and DHR

7.2 Function of Head regulator, Cross Regulator, Outlet, Drop and Escape

The regulators are required on a channel to regulate and control the supply of water. The functions of the distributary head regulator and the cross – regulator are summarized below.

7.2.1 Function of Distributary Head Regulator

A distributary head regulator serves the following main purposes.

1. It regulates the supply of water from the parent channel to the offtaking channel.
2. It controls the entry of silt into the offtaking channel.
3. It can serve as a meter for the measurement of discharge.
4. It is used for shutting off the supply into the offtaking channel when water is not needed, or when the offtaking channel is required to be closed for repairs or maintenance.

7.2.2 Function of Cross Regulator

1. The main function of cross regulator is to raise the water level in the parent channel in the upstream so that the offtaking channel can take its full supply even when the water level in the parent channel is lower than F.S.L.
2. It is also used to close the supply in the parent channel on its downstream. The supply in that case is usually directed to other channels. If an escape is also provided in conjunction with a cross regulator, the water can be directed to the escape channel.
3. There is usually a bridge on the cross regulator, which provides a means of communication.
4. It helps to absorb fluctuations in the various section of the canal system and thus prevents braches in the tail reaches.
5. It can be used to control discharge at an outfall of a canal into another canal or a lake.

6. It can be very easily combined with a canal fall: in which case, it helps to control the water surface slope for bringing the canal to a regime slope and to a balanced depth.
7. It can be sued to control the drawdown when the subsoil water levels are high to ensure safety of canal lining.
8. Cross regulators are useful for effective regulation of the entire canal system. In a good canal system, a large number of cross regulators are usually provided.

7.2.3 Function of Outlet (Modules)

A canal outlet or a modules is small structure built at the head of the watercourse so as to connect it with a minor or a distributary channel. Thus the water course takes water from the distributary channel at the outlet and carries it to the agricultural fields. An outlet for a water course serves the same purpose as a distributary head regulator serve for the distributary. Of course its size is much smaller. The importance of an outlet is considerable because it is a connecting link between the distributary maintained by the government and a water course maintain by farmers. The functions of the outlet are as follows:

1. To take water from minor or distributary channel.
2. It distributes water in required proportions in fields.
3. An out let may also be used for the measurement of discharge being supplied to the water course.

7.2.4 Function of Escapes

An escape is a structure constructed in irrigation channel for disposal of surplus water from the channel. Hence it is also called surplus water escape or canal surplus escape.

Function of escape:

1. It helps to overflow the extra surplus water from the canal safely.
2. It prevents the damage and de – function of irrigation canal due to flood water.
3. It prevents the damage of the farming land by spilling the excess water to streams.
4. It helps scouring of excess bed silt deposited in canals.

7.2.5 Function of Drop Structure

Whenever the available natural ground slope is steeper than the designed bed slope of the channel, the difference is adjusted by constructing vertical falls or drops in the canal bed at suitable interval.

Function of fall:

1. It lowers the bed level and water level of the channel to maintain the ground level.
2. It provides safety against erosion in bed and bank due to excess energy flow.

7.3 Design of Regulator and Escape

7.3.1 Design of Cross regulator and Distributary head regulator

Step 1: Design of crest level and length of water way

1. **Crest level**
   (a) The crest level of cross regulator is generally kept at the U/S bed level of the channels [Figure 7.5].

(b) The crest level of a distributary head regulator is generally kept 0.3m to 1m higher that the crest level of the cross regulator [Figure 7.6]

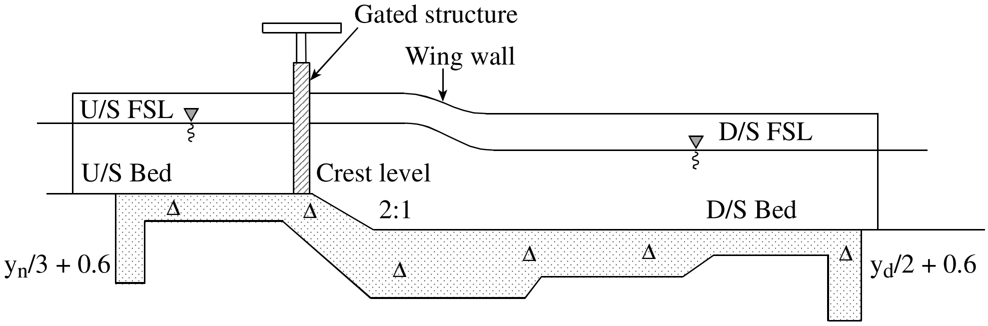


Figure 7.5: Cross Regulator

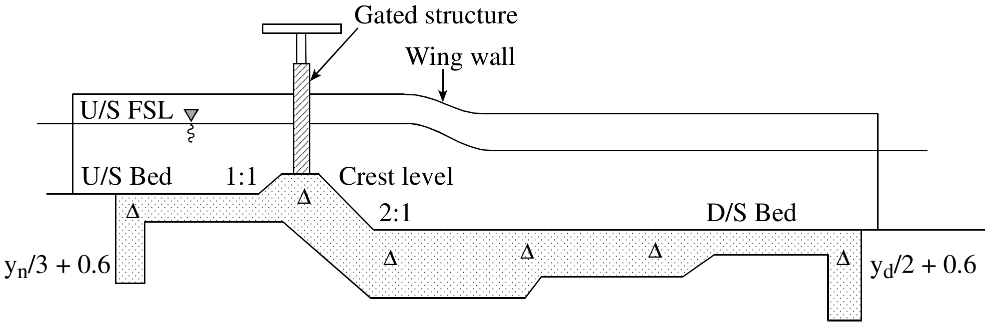


Figure 7.6: Distributary Head Regulator

2. Water way

(a) The length of water way is obtained by using the discharge formula for a drowned weir which is as follows:

$$Q = \frac{2}{3} Cd_1 \sqrt{2g}\, B\, [(h + h_v)^{3/2} - h_v^{3/2}] + Cd_2\, B\, h_1 \sqrt{2g\,(h + h_v)}$$

Where, $Cd_1 = 0.577$

$Cd_2 = 0.8$

Q = Discharge in cumecs

B = Clear water way

h = Difference of water level U/S and D/S of the crest in m

h_v = Head due to velocity of approach

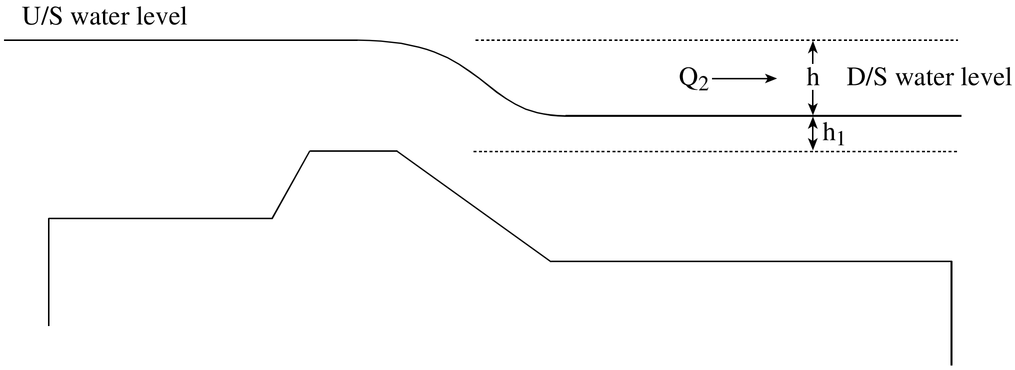
h_1 = Depth of D/S water level in the channel above the crest.

Generally h_v is small and can be neglected

$$Q = \frac{2}{3} \times 0.577 \sqrt{2 \times 9.81}\, B\, (h)^{3/2} + 0.8\, B\, h_1 \sqrt{2 \times 9.81\,(h)}$$

$$Q = 1.7\, B\, h^{3/2} + 3.54\, Bh_1\sqrt{h}$$

$$Q = B\sqrt{h}\,[1.7\, h + 3.54\, h_1]$$



Step 2: Design of impervious floor

1. **Level and length of D/S floor**

   The level and length of D/S floor is determined for the following two conditions.

   (a) Full supply discharge is passing through both the cross regulator and distributary head regulator with all gates fully open.

   (b) The discharge in the parent channel is insufficient but the offtaking channel is running full and its FSL is maintained by the partial opening of the gates of cross regulator.

   In first condition q and H_L are fixed; while in the second case q reduces and H_L increases, depending upon the low flow of the channel. The first condition become more critical.

   If the low flow record of parent canal is not given, the design can be done for the first flow condition.

2. **Downstream floor level or cistern level**

   For the above two flow conditions q an H_L are work out. Then Ef_2 is found from Blench Curve (Annex III). The level at which jump would form i.e. the level of D/S floor is then given by D/S TEL – Ef_2. ≈ D/S FSL – Ef_2

   If the D/S floor for the worst condition workout to be higher than the D/S bed level of the channel, the floor is provided at the bed level.

3. **Length of D/S floor**

   It is worked out by calculating

   (a) Length of jump consideration $L = 5(y_2 - y_1)$.

   (b) Exit gradient consideration[1] $=L = \frac{2}{3} b = \frac{2}{3} \alpha d$

   Where, b = Total length of impervious floor

   d = Downstream cut off depth.

   Length of D/S floor is provided whichever is greater than above conditions.

4. **Total floor length**

   The total floor length "b" is worked out form safe exit gradient consideration. This total floor length is than suitably distributed upstream and downstream.

[1] Refer Chapter 5

5. Thickness of impervious floor

The thickness of impervious floor is found from the consideration of uplift pressure. However, a minimum thickness of 0.3m to 0.5 m is provided from the practical consideration.

Step 3: Design of protection work

Cutoff piles are provided as given below:

1. Upstream cut off

The minimum depth of U/S cutoff below U/S floor level is kept as $(y_u/3 + 0.6)$ m, where y_u is the depth of water in the channel upstream.

2. Downstream cut off

The minimum depth of D/S cut off below the D/S floor level is kept as $(y_d/2 + 0.6)$m , where y_d is the depth of water in the channel downstream.

3. Launching apron

The C.C. (cubic content) blocks and inverted filter are provided in a length approximately equal to 1.5D. The quantity of stone in launching apron is kept as 2.25D m^3/m.

Example 7.1:

Design a cross regulator and Head regulator for a channel which takes off from the parent channel with the following data:

Discharge of parent channel = 105 cumecs

Discharge of distributary = 15cumecs

FSL of the parent channel, $\frac{U/S}{D/S} = \frac{118.10m}{117.90m}$

Bed width of Parent channel $\frac{U/S}{D/S} = \frac{45m}{41m}$

Depth of water in the parent channel $\frac{U/S}{D/S} = \frac{2.5m}{2.5m}$

FSL of distributary = 117.2m

Bed width of distributary = 15m

Full supply depth of distributary = 1.6m

Silt factor = 0.8

Assume safe exit gradient = 1/5

Solution:

A: Design of Cross regulator

Step 1: Design of crest and water way

Crest level = U/S bed level of the parent canal.

= FSL of parent canal – water depth

= 118.10m – 2.50m = 115.60 m

Water way: $Q = \frac{2}{3} Cd_1 \sqrt{2g}\, B\, [(h + h_v)^{3/2} - h_v^{3/2}] + Cd_2\, B\, h_1 \sqrt{2g\,(h + h_v)}$

Ignoring velocity head; $Cd_1 = 0.577$ and $Cd_2 = 0.8$

$Q = B \sqrt{h}\, [1.7\, h + 3.54\, h_1]$

Here h = U/S FSL – D/S FSL = 118.10 – 117.90 = 0.2m

h_1 = D/S FSL – Crest level = 117.90 – 115.60 = 2.3m

Now; $140 = B \sqrt{0.2}\, [1.7 \times 0.2 + 3.54 \times 2.3]$

Solving; B = 27.65m

Provide 4 bays of 7m each with a clear water way = 4 × 7 = 28 m

Provide 3 number of pier of 1.5m width each = 4.5 m

Thus the overall waterway = 28 + 4.5 = 32.5 m.

Step 2: Design of impervious floor

1. Level of impervious floor

$Q = 105\ m^3/sec.$

q = Q/B= 105/28 = 3.75 cumecs/m

H_L = U/S FSL – D/S FSL = 0.2

From Blench curve (Annex III); $Ef_2 = 1.95m$

(a) D/S floor level = D/S FSL – Ef_2 = 117.90 – 1.95 = 115.95m

(b) D/S bed level = D/S FSL – D/S flow depth = 117.90 – 2.5 = 115.40m

Hence provide the cistern or D/S floor at RL = 115.40m (Lower of a and b)

2. Length of D/S floor

(a) From Energy consideration

Length of D/S floor required = $5(y_2 - y_1)$

From Annex III for $Ef_2 = 1.95$; $y_2 = 1.7$ m

$Ef_1 = Ef_2 + H_L = 1.95 + 0.2 = 2.15m$; $y_1 = 0.7m$

Length = $5(y_2 - y_1) = 5\,(1.7 - 0.7) = 5m$

(b) From exit gradient consideration

$G_E = \frac{H}{d} \frac{1}{\pi\sqrt{\lambda}}$ Where; H = maximum static head, which is caused when there is full water on U/S and there is no water on D/S.

H = U/S FSL – D/S bed level = 118.10 – (117.90 – 2.50) = 2.7 m

d = D/S cutoff depth = $y_d/2 + 0.6 = 2.5/2 + 0.6 = 1.85m$

$$\frac{1}{5} = \frac{2.7}{1.85} \frac{1}{\pi\sqrt{\lambda}}$$

$$\frac{1}{\pi\sqrt{\lambda}} = 0.137\ ;\ \lambda = 5.3983$$

We have $\lambda = \dfrac{1 + \sqrt{1+\alpha^2}}{2}$

Solving we get $\alpha = 9.74$

$b = \alpha.d = 9.74 \times 1.85 = 18.03$ m say 18.1m

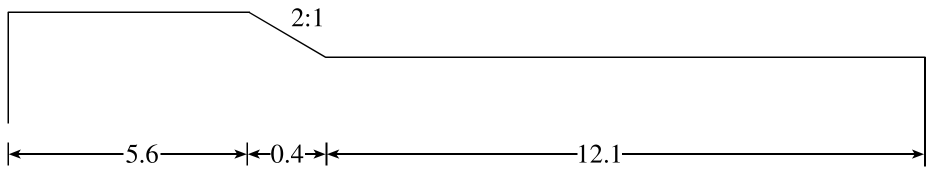
Minimum D/S floor length required $= \dfrac{2}{3} \times b = 12.1$m

Hence provide 12.1m as the D/S floor length

Provide sloping glacis with slope 2:1

Now glacis length = 2 × (115.60 – 115.40) = 0.4m

The balance, i.e. 18.1 – 12.1 – 0.4 = 5.6 m is provided as U/S floor length, as shown in figure below.



Step 3: Design of protection work

(a) U/S cutoff

The depth of U/S cutoff below U/S floor level = ($y_u/3 + 0.6$) = 2.5/3 + 0.6 = 1.43m ≈ 1.5m

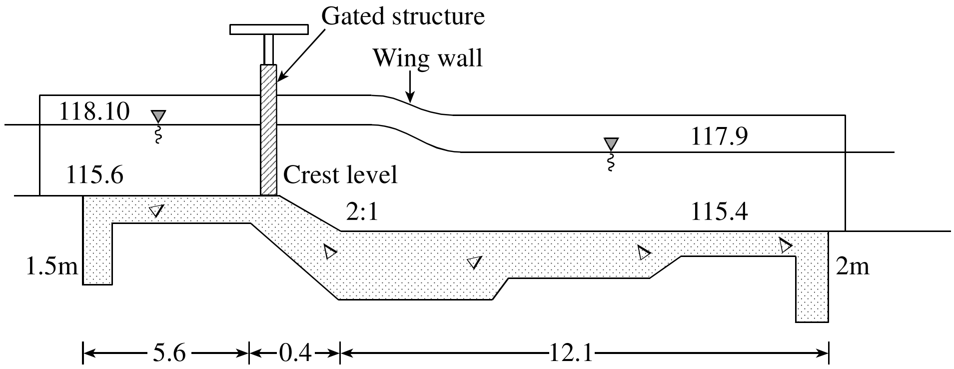
RL of bottom of U/S cutoff pile = 115.6 – 1.5 = 114.1m

(b) D/S cutoff

The depth of U/S cutoff below U/S floor level = ($y_d/2 + 0.6$) = 2.5/2 + 0.6 = 1.85m ≈ 2.0m

RL of bottom of U/S cutoff pile = 115.4 – 2.0 = 113.4 m

The floor thickness should be provided by Khosla's seepage theory as worked out in examples of chapter 5.



B: Design of Distributary head regulator

Step 1: Design of crest level and water way

Crest level = U/S bed level + 0.5 (say) = 115.6 + 0.5 = 116.1m

Water way

The discharge is given by $Q = B\sqrt{h}\,[1.7\ h + 3.54\ h_1]$

Where Q = 15 cumecs

h = FSL parent canal – FSL offtake canal = 118.10 – 117.20 = 0.9 m

h_1 = FSL offtake canal – crest level of DHR = 117.2 – 116.1 = 1.1m

$15 = B\sqrt{0.9}\,[1.7 \times 0.9 + 3.54 \times 1.1]$

B = 2.91 m

Which is very small as compared to bed width of distributary (15m).

Now provide 2 bays of 3.5m each with 1.0 m pier in between.

Overall water way is provided = 7 + 1= 8 m

Clear water way = 7 m.

Step 2: Design of impervious floor

Level of impervious floor

Q = 15 cumecs

q = 15/7 = 2.14 cumecs/m

H_L = U/S FSL – D/S FSL = 118.1 – 117.20 = 0.9m

From Blench curve (Annex III);

Ef_2 = 1.65 m

$Ef_1 = Ef_2 + H_L$ = 1.65 + 0.9 = 2.55m

From Annex III;

For Ef_2 = 1.65 m; y_2 = 1.56 m

Ef_1 = 2.55 m; y_1 = 0.32 m

D/S floor level = D/S FSL – Ef_2 = 117.2 – 1.65 = 115.55 m

D/S bed level = D/S FSL – D/S flow depth = 117.2 – 1.60 = 115.60m

Since the required D/S floor level is lower than the D/S bed level, the D/S floor level is kept as 115.50m

Length of impervious floor

1. **From Energy consideration**

Length of cistern required = 5 ($y_2 - y_1$) = 5 (1.56 – 0.32) = 6.2 m

2. From exit gradient consideration

$G_E = \frac{H}{d}\frac{1}{\pi\sqrt{\lambda}}$ Where; H = maximum static head, which is caused when there is full water on U/S and there is no water on D/S.

H = U/S FSL – D/S floor level = 118.10 – 115.50 = 2.6m

d = D/S cutoff depth = $y_d/2 + 0.6 = 1.6/2 + 0.6 = 1.4$ m adopt d = 1.85m (same as cross regulator)

$$\frac{1}{5} = \frac{2.7}{1.85}\frac{1}{\pi\sqrt{\lambda}}$$

$$\frac{1}{\pi\sqrt{\lambda}} = 0.137 \; ; \; \lambda = 5.3954$$

We have $\lambda = \frac{1+\sqrt{1+\alpha^2}}{2}$

Solving we get $\alpha = 9.74$

$b = \alpha.d = 9.74 \times 1.85 = 18.03$ m say 18.1m

Minimum D/S floor length required $= \frac{2}{3} \times b = 12.1$m

(i) Hence adopt D/S floor length = 12.1 m

(ii) Length of D/S glacis with 2:1 slope = 2 (116.10 – 115.50) = 1.2m

(iii) Width of crest = 1.0 m

(iv) Length of U/S glacis with 1:1 slope = (116.10 – 115.60) = 0.5 m

(v) Balance floor length to be provided on U/S side = 18.1 – (12.1 + 1.2 + 1 + 0.5) = 3.3 m.

Step 3: Design of protection work

(a) U/S Cutoff pile

The depth of U/S cutoff below U/S floor level = $(y_u/3 + 0.6) = 2.5/3 + 0.6 = 1.43\text{m} \approx 1.5\text{m}$

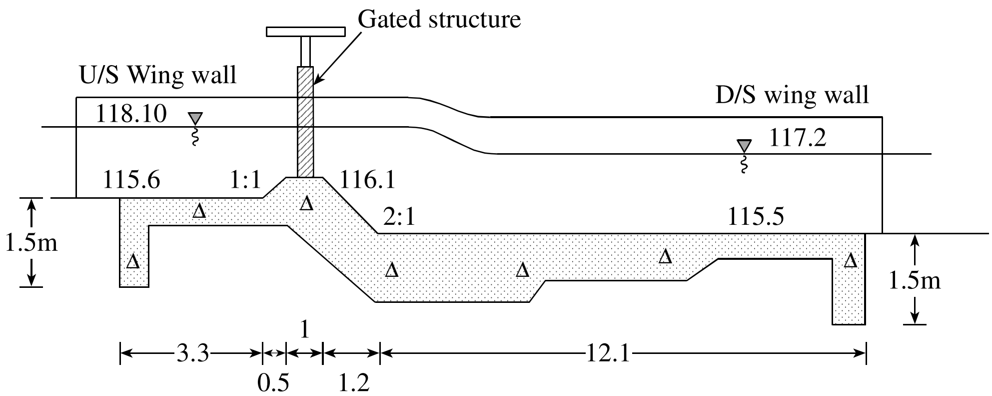
RL of bottom of U/S cutoff pile = 115.60 – 1.5 = 114.10 m

(b) D/S cutoff

The depth of U/S cutoff below U/S floor level = $(y_d/2 + 0.6) = 1.6/2 + 0.6 = 1.4$m adopt 1.5m.

RL of bottom of U/S cutoff pile = 115.5 – 1.5 = 114.0 m

The floor thickness should be provided by Khosla's seepage theory as worked out in examples of chapter 5.



7.4 Outlets

The canal outlet is a small structure built on the bank of a distributary channel through which water is supplied to a water course (or a field channel). In other words, the canal outlet is a structure provided at the head of the water course to connect it to the distributary channel. Thus the water course takes water from the distributary channel at the outlet and carries it to the agricultural fields.

An outlet for a water course serves the same purpose as a distributary head regulator serves for the distributary. Of course, its size is usually much smaller. The importance of an outlet is considerable because it is a connecting link between the distributary maintained by the department and a water course maintained by the cultivators.

An out let may also be used for the measurement of discharge being supplied to the water course. The discharge through an outlet is usually less than 85 lps. A canal outlet is also called an irrigation outlet or a module. The adjective canal in canal outlets is necessary to avoid the ambiguity with the dam outlets.

The design of a canal outlet is of great importance for the canal management as well as for the cultivators. The cultivators aim is usually to get an ample supply of water for the crops whereas the aim of the department is to ensure an equal distribution of water when the supply is limited. An outlet plays an important role in the equitable distribution of available water to all the cultivators.

7.4.1 Requirements of Good Canal Outlets

A good canal outlet should fulfill the following requirements.

1. It should be simple in design, construction and maintenance.
2. It should be quite strong and durable. It should not have any movable part which is likely to be easily damaged or which requires periodical maintenance.
3. The outlet should be such that it is not easily tampered with by the cultivators to increase the discharge in the water course. Moreover, if tampered with, it should be easily detectable and repairable.
4. It should be able to work efficiently and draw its full discharge even under a small working head.
5. It should draw its due share of silt.
6. It should not be expensive.
7. Its design should be such that it can be easily constructed by the local workers.
8. The design of the outlet should be such that it draws discharge according to the fluctuation of the discharge in the distributary. In other words, it should draw proportionately less water when the discharge in the distributary is less.

7.4.2 Types of Outlets

Outlets may be broadly classified into the following three types:

1. Non – modular outlet

A non – modular outlet is the outlet whose discharge depends upon the difference in water levels of both the distributary and the water courses. The discharge through a non – modular outlet, therefore, varies with the variation of water levels in both the distributary and the water course.

2. Semi – modular outlet (or flexible outlet)

A semi – modular out (or flexible outlet) is the outlet whose discharge depends only upon the water level in distributary and is independent of the water level in the water course. Thus the discharge in a semi – modular outlet doesn't depend upon the fluctuation in the water course, provided a minimum working head required for its working is available.

3. Modular outlet (or rigid module)

A modular outlet (or rigid module) is the outlet whose discharge is independent of water levels of both the distributary and the water course, provided a minimum working head required for its working is available other words, a modular outlet maintains a constant discharge irrespective of variation of water levels in the distributary and the water course.

7.4.3 Criteria for Selection of Outlet Capacity

The following criteria govern the selection of the capacity of an outlet.

1. Method of regulation of supply in the main canal

If the water supply in the main canal and the distributary is regulated according to the area actually irrigated in a crop season, the outlet capacity should be such that it can be easily changed from one crop season to the other. On the other hand, if the water supply is regulated according to the culturable Commanded area (CCA), the outlet should have a constant capacity. The latter method of regulation is considered more equitable and practical.

2. Method of distribution of water to cultivators

If all the cultivators share the outlet discharge simultaneously from a water course in proportion to their areas, there will be very high conveyance and application losses and the efficiency will be low. On the other hand, if the entire discharge of the water course is taken by each cultivator turn for a period proportional to his area, the losses will be small, and the outlet capacity can be decreased.

3. Sources of supply

If the supply to the canal is from a storage reservoir, the cultivators can be provided with water according to their need, provided the channel capacity is adequate. Outlets should be capable of supplying the required water. Moreover it should be possible to open or to close them whenever required. On the other hand, in case of the canal taking off from a diversion headworks with little storage, the water has to be supplied whenever it is available; otherwise, it will go waste. In that case, all the outlets generally remain open when the channel is running and there is no need of closing the.

4. System of working of the distributary

In the case of the channel taking off from a diversion headworks, the water available is generally not adequate to feed all the distributaries simultaneously to full capacity. In the period of keen demand, there are two alternatives;

i. All distributaries are run with low discharges simultaneously.

ii. Some of distributaries are run to full capacity and other are kept dry.

This latter system is preferred in the interest of efficient distribution of water. In the former case, the outlets should be able to take their proportionate share of discharge when there is a large variation in supply. In the latter case, the outlet should be able to take full discharge when the distributary is running and there is enough water in the distributary to feed all the outlets. No outlet is kept closed in this case, unless desired by the cultivators.

5. System of assessment

The out-let capacity is related to the method of assessment of the water supplied to the cultivators. If the assessment is based on the volume of water supplied or on the capacity of the outlet, the outlets should have a constant capacity. On the other hand if the assessment is based on the area irrigated, the outlet capacity should vary with the variation in the water levels of the distributary and the water course.

7.4.4 Design of Non-Modular Pipe Outlet (submerged outlet)

The most commonly used type or the outlet is the non – modular outlet, also called a pipe outlet. In its simplest form it consists of a pipe 10 cm to 30cm diameter embedded in the bank of the canal such that its exit is submerged below the water level in the water course. Thus the pipe outlet is a submerged pipe or the drowned pipe as Figure 7.7.

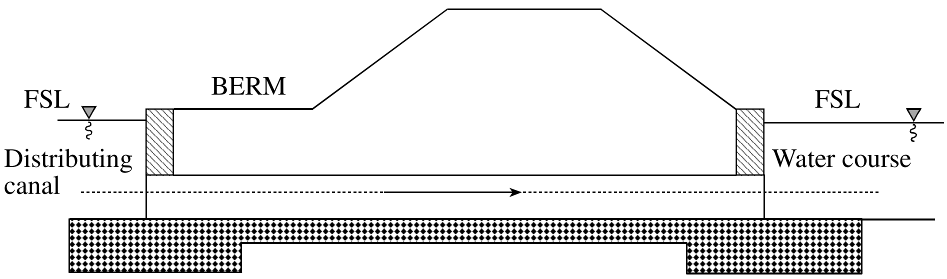


Figure 7.7: Non – modular outlets

The pipe is usually provided with face walls at its upstream and downstream ends. The pipe is laid on a concrete foundation of lean concrete to prevent uneven settlement and consequent leakage. The pipe is generally fixed horizontally at right angles to the direction of flow in the distributary. Its inlet is kept about 22.5 cm below the water level in the distributary. However, in the case of main canal or a branch canal, where considerable fluctuation in water level can take place the inlet is fixed at a suitable level taking into consideration the minimum water level likely to occur.

Because there is considerable disturbance at the entrance to the pipe, this type of outlet usually takes its due share of silt. If required, the silt conduction can be further improved by depressing the inlet of pipe and placing the pipe sloping upwards from the inlet to the exit at a slope not steeper than 1 in 12. It is the common practices to place the pipe inlet at a bed of distributary to enable the outlet to draw a fair share of silt carried by the distributary.

Sometimes, a non – modular outlet consists of a rectangular or circular opening with a pavement. It may also be an open sluice instead of a pipe.

Discharge formula

The discharge through a pipe (or barrel) non – modular outlet is determined by equating the difference (h) in the water levels in the distributary and the water course to the sum of losses at entry, due to friction and at exit. Thus,

$$h = 0.5\frac{V^2}{2g} + \frac{f\,L\,V^2}{2gd} + \frac{V^2}{2g} \qquad \ldots 7.1$$

Where h = difference in water levels of the distributary and the water course,

V = velocity through the pipe.

L and d = length and diameter of the pipe respectively.

From equation 7.1 $V = \sqrt{2gh}\left(\frac{d}{1.5d + f\,L}\right)^{1/2}$

The discharge q through the pipe outlet is given by

$$q = V\,a = a \times \sqrt{2gh}\left(\frac{d}{1.5d + f\,L}\right)^{\frac{1}{2}} \qquad \ldots 7.2$$

Where a is the cross sectional area of pipe.

For convenience, equation 7.2 may be written as

$$q = C\,a \times \sqrt{2gh} \qquad \ldots 7.3$$

Where C is the coefficient of discharge which depends upon the friction factor, length and diameter of pipe and is given by

$$C = \left(\frac{d}{1.5d + f\,L}\right)^{\frac{1}{2}}$$

The value of the friction coefficient ' f ' varies from 0.02 for clean pipe to 0.04 for slightly encrusted iron pipes. For earthen pipes, its value is usually taken as 0.03.

In general the values of C varies from about 0.61 to 0.51 for encrusted pipes of diameter 10 to 25 cm and of length of 3 to 15m. For an average value of 0.55, the discharge can be expressed as

$$q = 0.55\,a\sqrt{2gh} \qquad \ldots 7.4$$

The discharge through a non – modular outlet can be increased by deepening the water course bed and thereby increasing the value of h.

Example 7.2

Design a non – modular pipe outlet for the following conditions:

Discharge through the outlet = 0.02 cumecs

Length of the outlet = 15m

FSL of the distributary = 200.00

Available working head = 5cm

Coefficient of discharge = 0.51

Solution:

We have $q = C\ a \times \sqrt{2gh}$

$$0.02 = 0.51 \times \frac{\pi \times d^2}{4} \times \sqrt{2 \times 9.81 \times 0.05}$$

d = 0.2245m say 22.5cm.

Water surface level in the water course = 200.0 – 0.05 = 199.95m

Example 7.3

Design an irrigation outlet for the following data

Full supply discharge of outlet = 50 lps

FLS in distributing canal = 200.00m

FLS in water course = 199.92m

Full supply depth in distributing canal = 1.05m

Solution:

Available head across the outlet = FSL of distributary – FSL of water course = 200 – 199.92 = 0.08m

Since the available head is very small, a non – modular outlet (submerged pipe outlet) will have to be provided.

We have, $Q = C_d\ a \times \sqrt{2gh}$

Here Q =0.05 m^3/sec

C_d = 0.73 (assumed for submerge condition)

h = 0.08m

Now, $Q = C_d\ a \times \sqrt{2gh}$

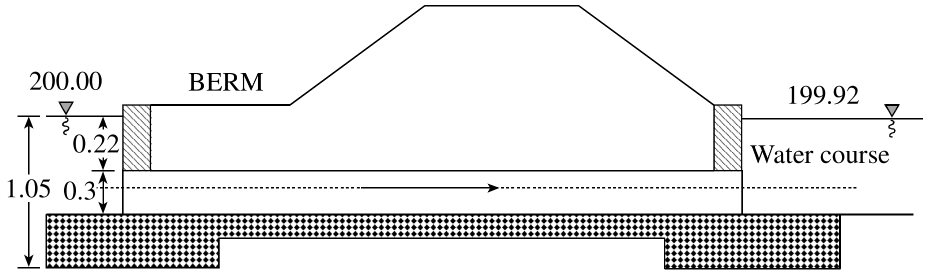
A = 0.055m^2

Diameter of pipe d = 0.264m

Hence, provide diameter of pipe = 30cm

The R.L. of the bed of the distributing canal = 200 – 1.05 = 198.95m

The pipe top can be fixed at about 22 cm below the FSL of distributing canal.



All dimensions are in m

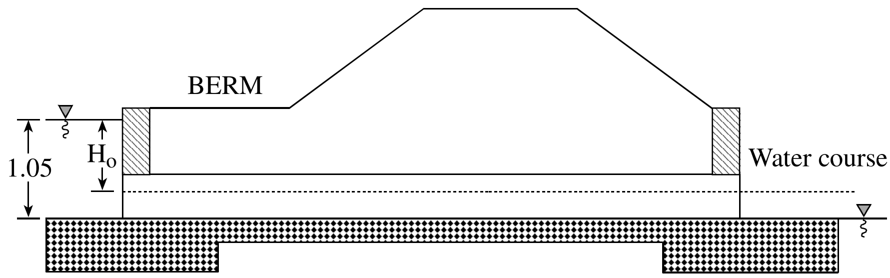
7.4.5 Design of semi-modular pipe outlet (free flow outlet)

Semi – modular outlets can be broadly classified into the following four types:

- Pipe outlet, discharging free
- Kennedy's gauge outlet
- Open – flume semi module
- Orifice semi – module

1. Pipe outlet, discharging free

The simplest type of a semi – modular outlet is a pipe outlet discharging free [Figure: 7.8]. In this case, the discharge is independent of water level in the water course, because the exit level is higher than the water level of the water course. The head (h) acting on the outlet is measured from the water level in the distributary to the center of the outlet of the pipe.



Water course

Figure 7.8: Pipe outlet (discharging freely)

The discharge through an outlet is given by $Q = C\,a\sqrt{2gH_o}$ . . . 7-5

Where C is the coefficient of discharge, a is the area of cross section of the pipe and H_o is the working head.

Example 7.4

Design a semi – modular pipe outlet for a discharge of 85 lps on a distributary with a full supply depth of 1.2m if the working head available is 0.70m. Take C = 0.62.

Solution:

From equation 7.5, $Q = C\,a\sqrt{2gH_o}$

$0.085 = 0.62\,a\sqrt{2 \times 9.81 \times 0.70}$

$a = 0.037\ m^2$

$\pi d^2/4 = 0.037$ or $d = 0.22m$

Let us select as 25cm ($0.049m^2$) diameter pipe.

Therefore, $0.085 = 0.62 \times 0.049\ \sqrt{2 \times 9.81 \times H_o}$

$H_o = 0.40m < 0.70$ m O.K.

The outlet will be a semi – modular outlet because there is free falling flow i.e. pipe outlet is higher than the water level in the water course.

Example 7.5

Design a pipe outlet for the following data:-

Full supply discharge at the head of the watercourse = 90 lps

FSL in distributing canal = 205.0m

FSL in water course = 204.0m

Solution:

Available head across the outlet = FSL of distributary – FSL of watercourse

= 205.0 – 204.0 = 1m

Coefficient of discharge $C_d = 0.62$

Full supply discharge $Q = 0.09\ m^3/sec$

Now, the discharge through such an outlet is given as $Q = C\,a \times \sqrt{2gH_o}$

Assuming the diameter of pipe = 25cm

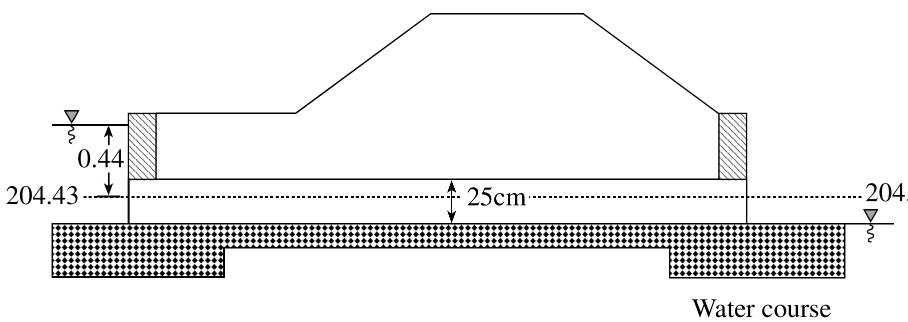
Now, $0.09 = 0.62 \times \frac{\pi \times 0.25^2}{4} \times \sqrt{2gH_o}$

$H_o = 0.44m$

R.L. of center of outlet of pipe = 205 – 0.44 = 204.56m

R.L. of bottom level of pipe = 204.56 – 0.25/2 = 204.43m which is greater than FSL of water coarse i.e. 204.00m

Hence, a pipe of 25cm diameter can be laid horizontal with its bottom or sill level at R.L. 204.43m and it will be discharging freely as a semi – module.



7.4.6 Rigid Modules

A rigid module gives a constant discharge irrespective of the variation in the water levels of the distributary and the water course. Modular outlets are basically of two types:

1. Modules with moving parts
2. Modules without moving parts

1. Modules with moving parts

The modules with moving parts are difficult to design and construct and are quite expensive. These modules are liable to frequent breakdowns due to increase in friction, rusting of the moving parts and clogging by silt, debris and weeds.

Various types of modules with moving parts are as under:

i. Visvesvaraya's self – acting module
ii. Kennedy's outlet module
iii. Kent's O – type module
iv. Khanna's auto – adjusting orifice distributor

Because of high initial cost and difficult in proper maintenance, the modules with moving parts are rarely used in practice.

2. Module without moving parts

The module without moving parts are relatively easy to design and construct and their maintenance cost is also low. In these modules, the discharge is automatically regulated by the velocity of water without the necessity of any moving part. These modules are quite good and are commonly used wherever a rigid module is required.

Various types of modules in this category are as follows:

i. Gibb's module   ii. Khanna's rigid module   iii. Ghafoor's rigid module

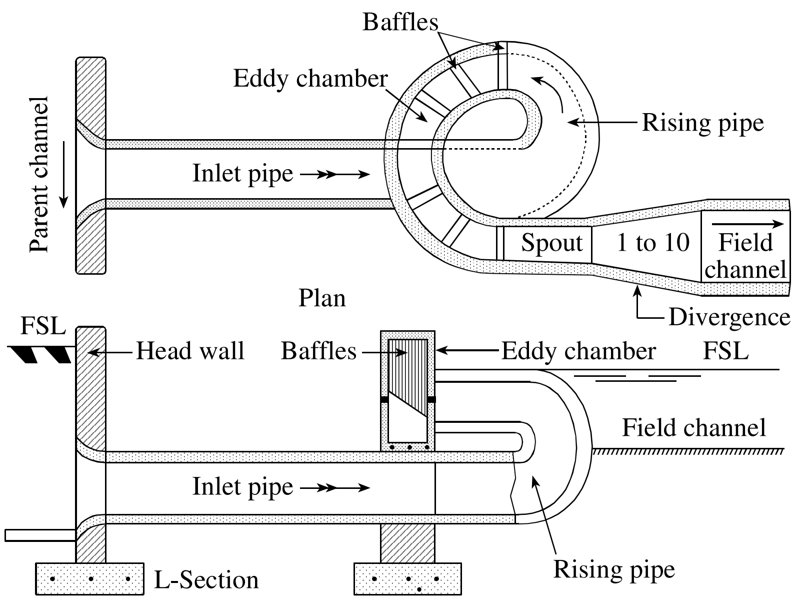


Figure 7.9: Gibb's Module

7.5 Drop or Fall

A drop (or fall) structure is a regulating structure which lowers the water level along its course. The slope of a canal is usually milder than the terrain slope as a result of which the canal in a cutting at its headwork will soon outstrip the ground surface. In order to avoid excessive infilling the bed level of the downstream canal is lowered, the two reaches being connected by a suitable drop structure Figure 7.10.

The drop is located so that the fillings and cuttings of the canal are equalized as much as possible. Wherever possible, the drop structure may also be combined with a regulator or a bridge. The location of an offtake from the canal also influences the fall site, with off-takes located upstream of the fall structure.

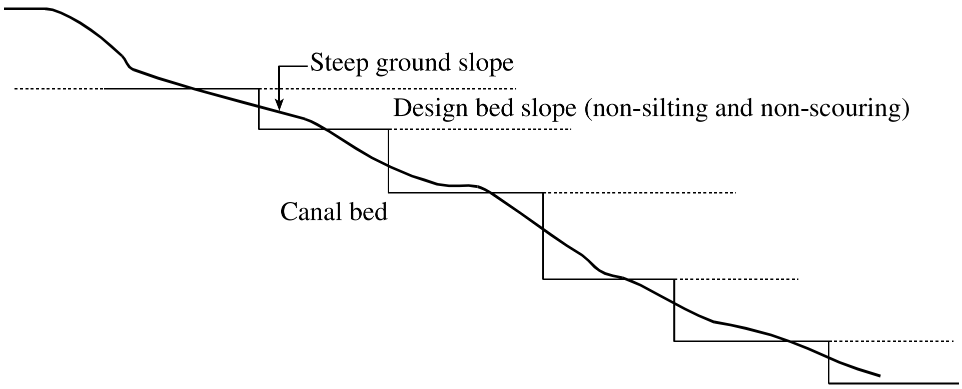


Figure 7.10: Location of Canal Drop

7.5.1 Types of Fall

The common different types of falls took place in various stages, as discussed below:

1. **Ogee – type fall**

   The ogee fall was evolved by Sir Proby Cautley. This fall was designed with the aim to provide a smooth transition from the upstream to the downstream water level and avoid disturbance and impact as far as possible (Figure 7.11). This fall is provided with gradual convex and concave curves. However, these falls had the following shortcomings:

   - On account of heavy drawn on the upstream side of the fall there was erosion of bed and banks of the channel.
   - Due to smooth transition the kinetic energy was almost fully preserved which caused erosion of bed and banks of the channel on the downstream side of the fall.

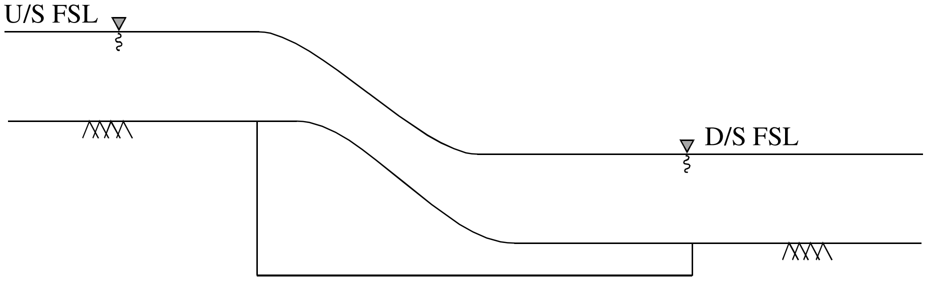


Figure 7.11: Ogee type fall

2. **Rapid fall (or rapid)**

The rapid fall or rapids was evolved by R.F. Croften. As shown in Figure 7.12, the rapids falls were constructed with boulder facing and were provided with glacis having gentles slopes in the range of 1 in 10 to 1 in 20. The long glacis assured the formation of hydraulic jump and hence these falls worked quite satisfactorily. Further these falls also admitted of timber traffic. However, the cost of construction of these falls was very high.

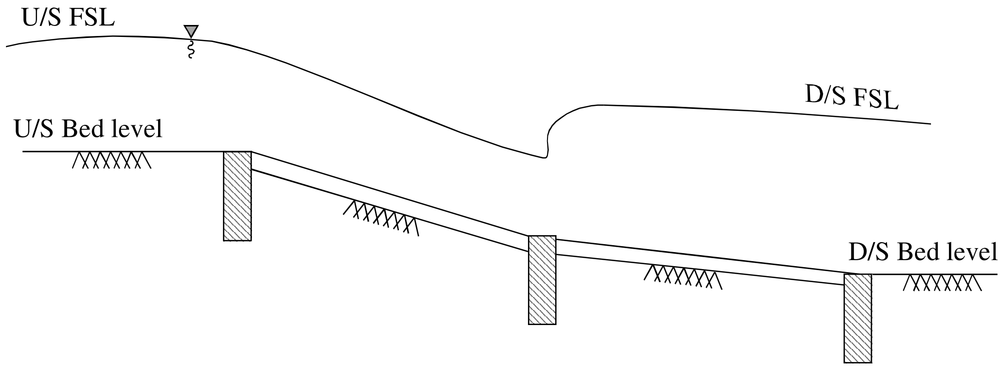


Figure 7.12: Rapid fall

3. **Stepped fall (Cascade)**

The stepped falls were the modified form of rapid falls in this respect that the long glacis of the rapid falls were replaced by floor in steps in the stepped falls as shown in Figure 7.13. However, the cost of construction of the stepped falls was also very high.

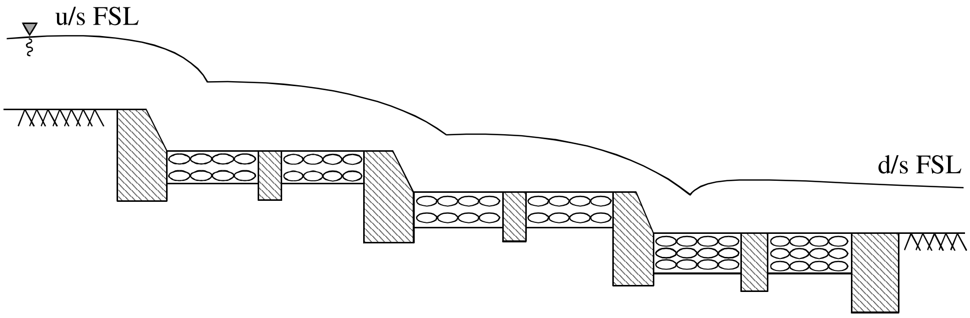


Figure 7.13: Stepped fall

After the development of stepped falls it was recognized that better dissipation of energy could be achieved through vertical impact of falling jet of water on the floor. As such vertical falls with cistern were evolved. However, the earlier types of vertical falls were not well developed and gave trouble. As such there were superseded for some time by trapezoidal notch falls.

4. **Trapezoidal notch fall**

The trapezoidal notch fall was evolved by Reid in 1894. This consists of a number of trapezoidal notched in a high breast wall across the channel with a smooth entrance and a flat circular lip projecting downstream from each notch to disperse water as shown in Figure 7.14. The notch were designed to maintain the normal depth of flow in the upstream channel at any two discharge values, the variation at intermediate values beings small. Thus the depth discharge relationship of the channel remained practically undisturbed by the introduction of the fall i.e. there was neither drawdown nor heading up of water in the channel upstream of the fall.

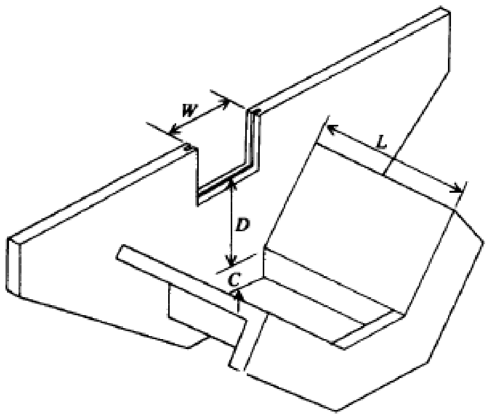


Figure 7.14: Trapexoidal Notch fall

5. Vertical drop fall

In the vertical drop type fall the clear nappe leaving the crest is made to impinge into a cistern below. The cistern provides a water cushion and helps to dissipate the surplus energy of the falling jet. In the earlier types of vertical drop type falls the dimensions to the cistern were arbitrarily decided on the basis of the past experience of the designers. Further a grid consisting of baulks of timber spaced a few centimeters apart was provided in the cistern to intercept the falling nappe and dissipate its surplus energy. This ware later abandoned because the timber grid got clogged with floating debris and rotted thus necessitating its frequent replacement.

6. Straight glacis fall

This type of fall utilize hydraulic jump (or standing wave) for the dissipation of energy. However, there was serious trouble with some of these falls. One of the causes of trouble was that even after the formation of hydraulic jump there was considerable surplus energy in water. Another cause of trouble was due to rapid expansion after fluming eddied ware developed which caused deep scours. Thus further research was carried out to eliminate these defects and two types of falls viz. Montague and Inglis type falls were evolved. In the Montague type fall the straight glacis has been replaced by a parabolic glacis commonly known as Montague profile. The Inglis type fall straight glacis is provided but at a certain distance from the toe of the glacis of baffle wall of certain height is provided. Both these types of falls have been designed with the aim to hold the hydraulic jump stable on or close to the glacis and thus affect maximum dissipation of energy.

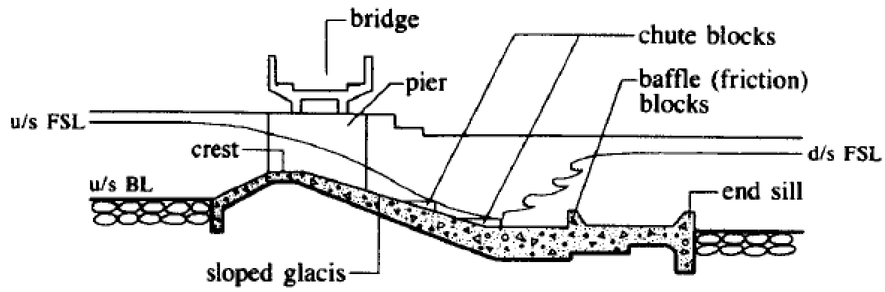


Figure 7.15: Sloping glacis type fall with USBR type III stilling basin

7. Common (straight) drop

The common drop structure, in which the aerated free – falling nappe (modular flow) hits the downstream basin floor, and with turbulent circulation in the pool beneath the nappe contributing to energy dissipation, is shown in Figure 7.16.

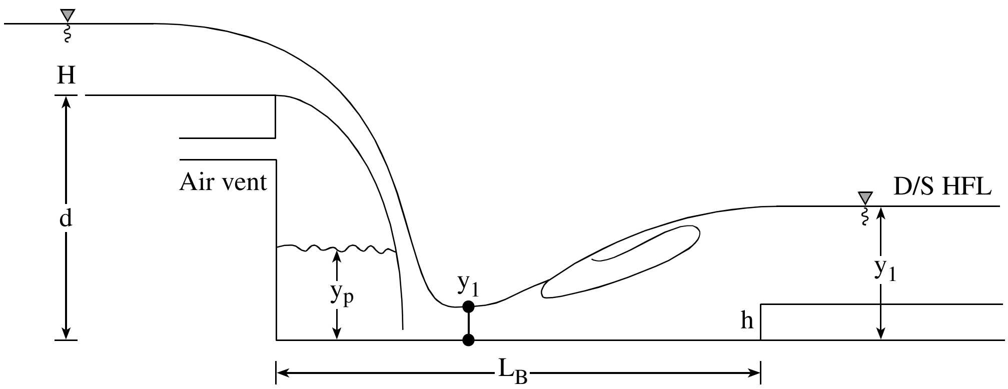


Figure 7.16: Common Drop structure (after Bos, 1976)

8. YMGT – type drop (Japan)

This type of drop is generally used in flumed sections suitable for small canals, field channels, etc., with discharges up to 1 m^3/sec (Figure. 7.17). The following are the recommended design criteria:

- Sill height, P varies from 0.06 m to 0.14 m with the unit discharge q between 0.2 and 1.0 m^3/sec /m;
- Depth of cistern, d_c=1/2(E_c H_{dr})$^{1/2}$
- Length of cistern, Lc = 2.5L_d

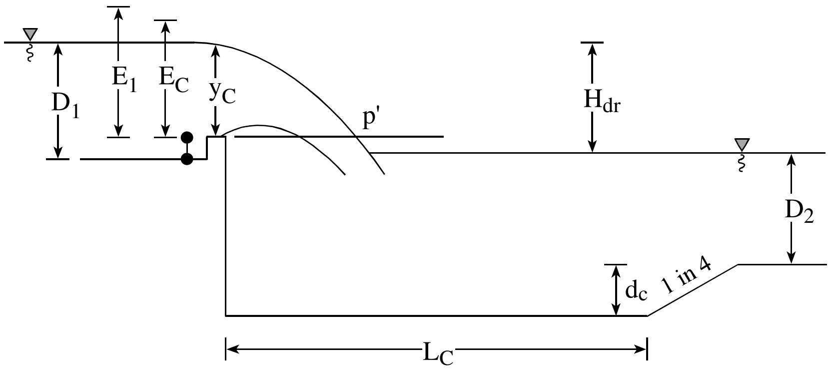


Figure 7.17: YMGT – type drop (Japan)

7.5.2 Design of vertical drop (Sarda type drop)

The Sarda – type fall is a vertical drop fall in which there is a raised crest and there is a vertical impact. This type of fall was evolved to replace the notch fall on the Sarda canal system in U.P. India. It was found to be more economical than the notch fall. Moreover, it was simple in design and construction. The maximum height of drop was 1.80m (6 ft.) In the commanded area of the Sarda canal, it was found that there was a thin stratum of sandy clay overlaying a stratum of pure sand. The depth of cutting was therefore kept small to avoid excavating the pure sand stratum and increasing seepage losses. For such conditions, instead of providing fewer falls with large drops, a large number of small falls were preferred. In the earlier designs, no depressed cistern was provided, and the downstream wings were not flared.

The complete design of the Sarda type fall consists mainly of the design of the following components:

1. Crest wall
2. Cistern
3. Impervious floor
4. D/S bed protection works
5. U/S wing walls
6. D/S wing walls
7. D/S side protection

Design of Sarda type fall

Step 1: Design of Crest

Length of the crest

The crest length is normally kept equal to the bed width of the canal; however, an increase in length by an amount equal to the flow depth takes into account any future increase in discharge.

Thus Length of crest L = Bed width of channels (U/S)

Shape of the crest

For discharge up to 14 m^3/sec	Rectangular shaped crest wall
For discharge > 14 m^3/sec	Trapezoidal shaped crest wall

1. For Rectangular crest wall

Top width $B = 0.55\sqrt{d}$

B_t = top width of crest wall

d = height of the crest wall above the D/S bed level.

Base width of crest wall $B_1 = \dfrac{h + d}{G}$

B_1 = base width of crest wall (m)

h = Height of water surface above the top of the crest

G = Sp. Gr of material of crest wall

Discharge $Q = Cd\sqrt{2g}\, L\, H^{3/2}\left(\dfrac{H}{B_t}\right)^{\frac{1}{6}}$ . . . 7.6

Where Q = discharge cumecs.

L = length (m)

H = head over the crest (m)

B_t = top width of crest wall (m)

Cd = coefficient of discharge 0.415

Now, equation 7.6 becomes,

$$Q = 1.84\, L\, H^{3/2}\left(\frac{H}{B_t}\right)^{\frac{1}{6}}$$

2. For trapezoidal crest wall

Top width of crest wall $B = 0.55\sqrt{H + d}$

Where, H = head over the crest

d = Height of crest wall above the D/S bed level in m.

Discharge $Q = C_d\sqrt{2g}\, L\, H^{3/2}\left(\dfrac{H}{B_t}\right)^{\frac{1}{6}}$ . . . 7.7

Where Q = discharge cumecs.

L = length (m)

H = head over the crest (m)

B_t = top width of crest wall (m)

C_d = coefficient of Discharge = 0.45

Now, Equation 7.7 becomes,

$$Q = 1.99\ L\ H^{3/2} \left(\frac{H}{B}\right)_t^{\frac{1}{6}}$$

Crest level

Crest level = U/S TEL – Head over the Crest

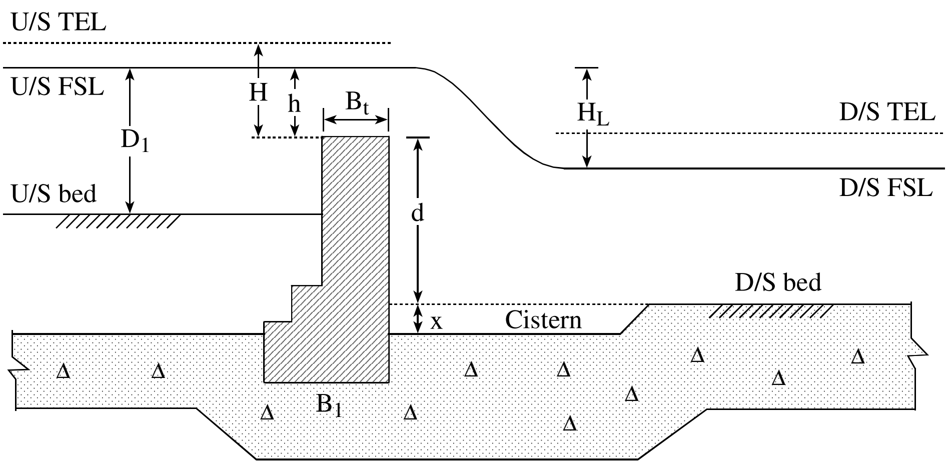
While neglecting velocity head

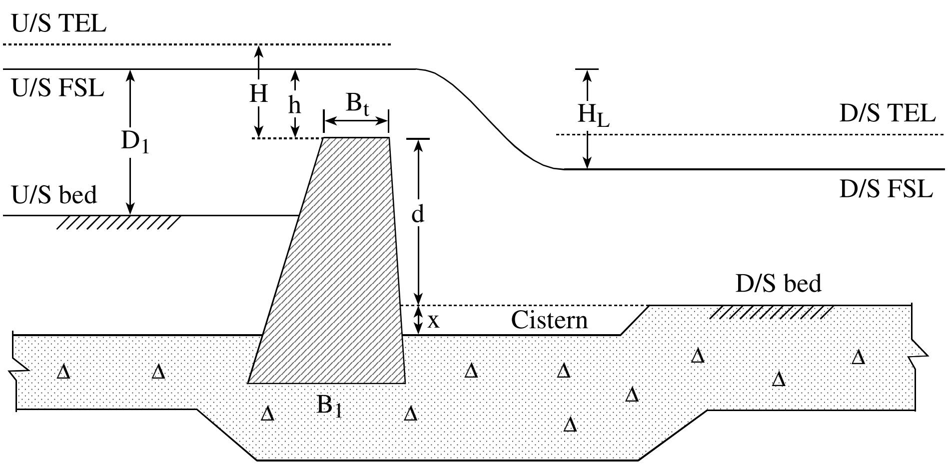
Crest level = U/S FSL – Head over the Crest

Head over the crest is obtained from (7.6) or (7.7) depend on shape of crest wall.

Height of crest wall

Height of crest wall = D_1 – h. refer figure.





Step 2: Design of cistern

Length of cistern $L_c = 5\sqrt{H \times H_L}$

Depth of cistern $x = \frac{1}{4}(H \times H_L)^{2/3}$

Neglecting the velocity head, H may be replaced by h in these equations.

Step 3: Design of impervious floor

- The length and thickness of impervious floor may be determined by Bligh's theory for small and medium fall and by Khosla's theory for larges falls.
- Maximum seepage head when there is water on the U/S side upto the top crest wall and there is no flow on downstream side.
- Maximum seepage head = d
- Hydraulic gradient for Bligh's theory = 1/5 to 1/8
- Total floor length according to Bligh's theory = $C \times H_s$ (where H_s is maximum seepage head in our case $H_s = d$)
- Out of the total length of the impervious floor a minimum length to be provided on the D/S of the crest wall l_d = 2(water depth + 1.2) + drop
- Floor thickness; Upstream thickness = 0.3m (minimum).
- D/S thickness = 0.3 – 0.4m for small fall.

  = 0.4 – 0.6m for large fall.

Step 4: Cut off or curtain wall

(a) U/S cutoff depth = $D_1/3$ (minimum)

(b) D/S cutoff depth = $D_2/2$ (minimum)

Where, D_1 = U/S flow depth

D_2 = D/S flow depth

Example 7.6

Design a vertical drop structure for the data given below.

Full supply discharge U/S and D/S = 1.8 cumecs.

Drop height = 0.75m

FSL U/S and D/S = 106.997 and 106.247

Full supply depth U/S and D/S = 0.929m

Bed levels U/S and D/S = 106.068 and 105.318

Bed width U/S and D/S = 1.2m

Top width of crest = 0.5m for initial assumption; C_d = 0.415 for rectangular crest. The drop structure is of masonry with specific gravity 2.0. Side slope of the canal is 1:1. The Bligh's coefficient is 6.0 for sandy loam soil at foundation.

Solution:

Step 1: Design of Crest

Length of the crest = D/S bed width of channel = 1.2m

Since rectangular crest is provided. The discharge formula is given by

$$Q = C_d \sqrt{2g}\, L\, H^{3/2} \left(\frac{H}{B_t}\right)^{\frac{1}{6}} \text{ For } C_d = 0.415 \text{ the equation becomes;}$$

$$Q = 1.84\, L\, H^{3/2} \left(\frac{H}{B_t}\right)^{\frac{1}{6}}$$

Here, Q = 1.8 cumecs.

L = 1.2m

For initial assumption top width B = 0.5m

Now $H^{5/3} = 0.7263$

H= 0.825m say 0.83m

Now approach velocity $V_a = \frac{\text{Discharge}}{\text{area}} = \frac{1.8}{(1.2 \times 0.929 + 1 \times 0.929^{2})} = 0.91$ m/sec

$V_a^2/2g = 0.0422$m

U/S TEL = U/S FSL + $V_a^2/2g$ = 106.997 + 0.0422 = 107.039m

RL of crest = U/S TEL – H = 107.039 – 0.825 = 106.214m

Height of crest above U/S floor = 106.214 – 106.068 = 0.146m

Adopt crest width at top $B_t = 0.55\sqrt{d}$

Where d = height of crest above D/S bed = 106.214 – 105.318m = 0.896m

Now top width $B_t = 0.55\sqrt{0.896} = 0.52$m

Keep 0.55m width of the crest.

Thickness at base $B_1 = \frac{h + d}{G} = \frac{0.825 + 0.896}{2} = \mathbf{0.86m}$

Step 2: Design of cistern

Length of cistern $L_c = 5\sqrt{H \times H_L}$

Where, H = Head over the crest + $V^2/2g$ = 0.825m

H_L = U/S FSL – D/S FSL = 106.997 – 106.247= 0.75m

Now length of cistern L_c = 4.0m

Adopt length of cistern L_c = **4.5m**

Now, Depth of cistern $x = \frac{1}{4}(H \times H_L)^{2/3} = \frac{1}{4}(0.825 \times 0.75)^{2/3} = 0.181$m

RL of bed cistern = RL of D/S bed – x = 105.318 – 0.181 = 105.137m

Step 3: Design of impervious floor

Seepage head $H_s = d = 0.896m$

Bligh's Co – efficient = 6.0

Required creep length = 6.0 × 0.896 = 5.376m

U/S cut off pile depth $d_1 = D_1/3 = 0.929/3 = 0.31m$

D/S cut off pile depth $d_2 = D_2/2 = 0.929/2 = 0.465m$

Provide, U/S cut off pile Depth d_1 = **0.5 m**

D/S cut off pile Depth d_2 = **0.75m**

Thus, vertical length of creep = 2(0.5+0.75) = 2.5m

Required length of horizontal impervious floor = 5.376 – 2.5 = 2.867m

Now, minimum length of impervious floor to be provided on the D/S of the crest wall

$L_d = 2(D_1 + 1.2) + H_L = 2(0.929 + 1.2) + 0.75 = 5.008m$

Now provide, D/S impervious floor = **5.5m**

Theoretically, there is no need of U/S floor (2.876 – 5.5 = –2.624m), but necessary to provide nominal length of U/S floor. For safety due to surface flow condition, provide U/S floor Length = 2m.

Step 4: Calculation of uplift pressure and thickness

Total creep length = 2 + 0.86 + 5.5 + 2(0.5 + 0.75) = 10.86 m

- The uplift pressure under the U/S floor will be counterbalanced by the weight of water and hence no thickness is required. However, provide a minimum thickness of 0.3m.
- For other points the minimum vertical ordinate between the Bligh's HGL and floor level gives the uplift pressure.
- Thus maximum unbalanced head under the D/S toe of the crest wall

$$= x + \frac{(\text{Crest level} - \text{D/S bed level})}{\text{Total impervious length}} \times \text{Length from D/S pile to required location}$$

$$= 0.181 + \left(\frac{106.214 - 105.318}{10.86}\right) \times 5.5 = 0.635m$$

Now, required thickness $t = \frac{h}{G-1} = \frac{0.635}{2.24-1} = 0.5125$

Adopt 0.52m thick cement concrete floor over laid by 0.2m brick pitching.

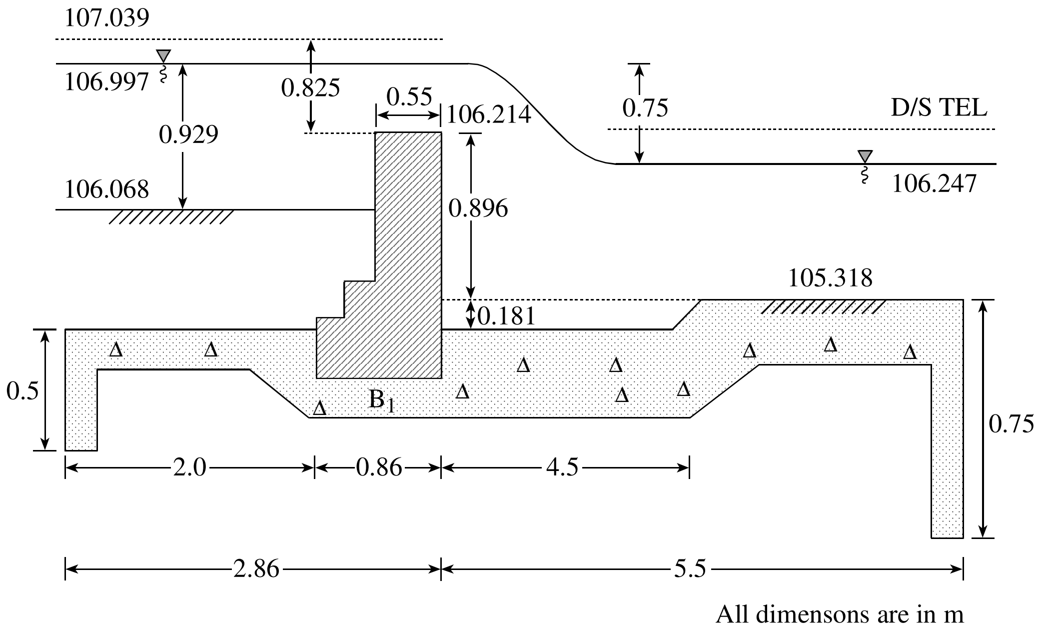


Figure 7.18: Designed vertical drop

Example 7.7

Design a Sarda type fall for the following data:

- Full supply discharge : $\frac{U/S}{D/S}$ = 45 cumecs
- Full supply level : $\frac{U/S}{D/S} = \frac{118.30m}{116.80m}$
- Full supply depth : $\frac{U/S}{D/S}$ = 1.8 cumecs
- Bed width : $\frac{U/S}{D/S}$ = 28m
- Bed level : $\frac{U/S}{D/S} = \frac{116.5m}{115.0m}$
- Drop : 1.5m

Design the floor on the basis of Bligh's theory taking coefficient of creep = 8. Check the design by Khosla's theory and modify the design if necessary. Safe exit gradient may be taken as 1/5.

Solution:

Step 1: Calculation of H and d

Since the discharge is more than 14 cumecs, trapezoidal crest will be provided for which discharge is given by equation $Q = 1.99\, L\, H^{3/2} \left(\frac{H}{B_t}\right)^{\frac{1}{6}}$ . . . (i)

Q = 45 cumecs

L= length of crest = bed width of the channel = 28m

The top width of crest wall $B_t = 0.55\sqrt{H + d}$

But $H + d = D_1$ + drop in bed level

$H+d = 1.8+1.5 = 3.3m$……………. (ii)

$B_t = 0.55\sqrt{3.3} = 1.0m$ ………… (iii)

Substituting the values in equation (i), we get

$$45 = 1.99 \times 28 \times H^{3/2}\left(\frac{H}{1}\right)^{\frac{1}{6}}$$

$H = 0.88m$

Step 2: Design of crest wall

Top width of the crest wall as calculated in step 1 is B = 1.0m

Provide, slope of U/S face = 1 in 3

Slope of downstream face = 1 in 8

Assuming channel side slope 1:1

Velocity approach $V_a = \frac{Q}{\text{Cross – sectional area of canal}} = \frac{45}{(28 + 1.8)1.8} = 0.84$ m/sec

Velocity head, $\frac{V_a^{\ 2}}{2g} = 0.036m$

U/S TEL = U/S FSL + $\frac{V_a^{\ 2}}{2g}$ = 118.30 + 0.036 = 118.336m

Crest level = U/S TEL – H = 118.336 – 0.88 = 117.456 m

Step 3: Design of cistern

We have the length of cistern $L_c = 5(H \times H_L)^{1/2}$

$$L_c = 5(0.88 \times 1.5)^{1/2} = 5.74m$$

Thus provide cistern of length 6m

Depth of cistern $x = \frac{1}{4}(H \times H_L)^{2/3}$ =0.30m

Thus provide cistern of depth 0.31m

R.L. of bed of cistern = RL of D/S bed – x = 115.00 – 0.31= 114.69m

Step 4: Design of impervious floor

Seepage head H_s = d = 117.456 – 115.00 = 2.456

Bligh's coefficient C = 8

Required creep length = 8 × 2.42 = 19.648m

Provide U/S cutoff of depth $d_1 = \frac{D_1}{3} = \frac{1.8}{3} = 0.6 \sim 1.0m$

And D/S cutoff of depth $d_2 = \frac{1.8}{2} = 0.9 \sim 1.5m$

Thus vertical length of creep = 2 (1.0 + 1.5) = 5.0m

Length of horizontal impervious floor = 19.648 – 5.0 = 14.648m

Provide 15m length of impervious floor.

Minimum length of impervious floor to be provided on the D/S of the crest wall

$L_d = 2(D_2 + 1.2) + H_L = 2(1.8 + 1.2) + 1.5 = 7.5m$

Provide L_d = 8m. The balance of the length = 15 – 8 = 7m in provided under and U/S of the crest wall.

Calculation for uplift pressure and thickness

Total creep length = 15 + 2(1.0 +1.5) = 20m

– The uplift pressure under the U/S floor will be counter balanced by the weight of water and hence no thickness is required. However, provide a minimum thickness of 0.4m.

– For other points the minimum vertical ordinate between the Bligh's HGL and the floor level gives the uplift pressure.

– Thus maximum unbalanced head under the D/S toe of the crest wall $=2.42\left(1-\frac{7+2\times 1.0}{20}\right)+x = 1.64\text{m}$

Required thickness $= \frac{h}{G-1} = \frac{1.64}{2.24-1} = 1.32\text{m}$

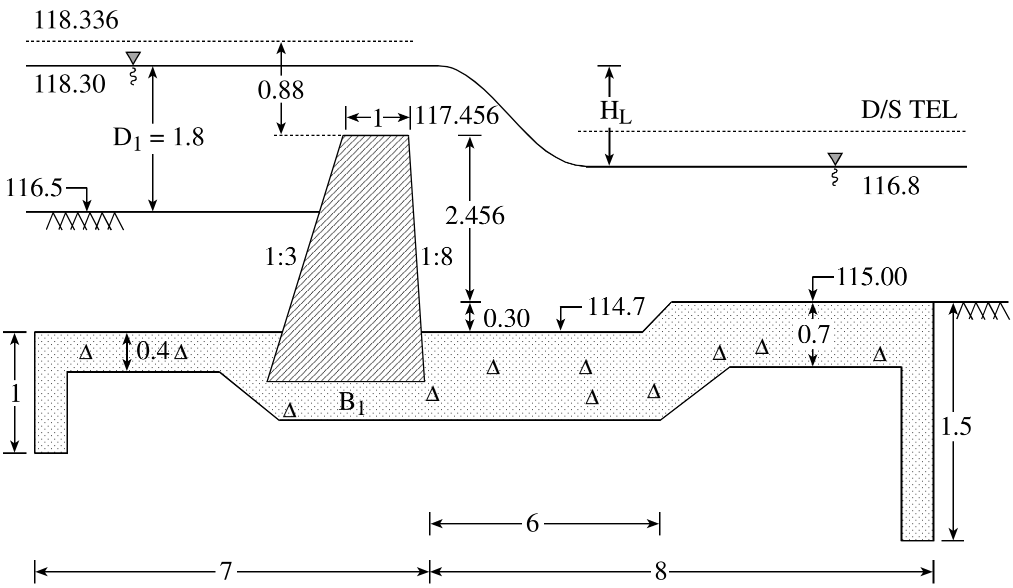
Provide 1.4m thick cement concrete floor over laid by 0.2m brick pitching.

– Provide minimum thickness of 0.6m overlaid by 0.2 m thick brick pitching at the D/S end of the floor.

– The thickness of the floor at intermediate points may be varied as per requirements of uplift pressure.

Step 5: Checking of floor thickness by Khosla's theory

The floor thickness should be checked by Khosla's seepage theory as worked out in examples of chapter 5.



All dimensons are in m

Exercise

1. Write down the function of head regulator and cross regulator.
2. Write the stepwise design procedure of cross regulator and distributary head regulator with supporting sketches.
3. What is the provision of drop structures required in a canal irrigation system? Explain with appropriate sketches.
4. What are the functions of the canal outlets?
5. What are the different types of good outlets? Give one example of each.
6. What are the criteria for selection of the outlet capacity?
7. Explain the working principle of non – modular and semi modular outlet. What are the requirement of a good module?
8. Design a semi – modular outlet with a discharge of 0.06 cumecs on a distributary with a full supply depth of 1m and the working head of 0.55m.
9. Design an open flume outlet with a discharge of 0.07 cumecs on a distributary channel with a full supply depth of 1.0m. The available working head is 0.20m.
10. Design the crest and cistern of a vertical drop structure for the data given below.

    Discharge = 4.5 cumecs, bed level U/S = 105.00, side slope of cannel = 1:1, bed level d/s = 103.5, FSL U/S = 106.5, bed width U/S and D/S = 3.0m, top width of crest = 0.75m (for initial assumption), Cd = 0.41.
11. Design a vertical drop for the following data;
    - Full supply discharge = 20 cumecs
    - Full supply level $\frac{US}{DS} = \frac{215.4m}{214.4m}$
    - Full supply depth $\frac{US}{DS} = \frac{2.0m}{2.0m}$
    - Bed width $\frac{US}{DS} = \frac{20m}{20m}$
    - Bed level $\frac{US}{DS} = \frac{213.40m}{212.40m}$
    - Drop height = 1.0 m
12. Design the floor on the basis of Bligh's theory taking coefficient of creep = 10. Check the design by Khosla's theory and modify the design if necessary. Safe exit gradient may be taken as 1/5. Sketch the longitudinal section of fall.

✸✸✸

CHAPTER 8

Cross-Drainage Structures

In an irrigation project, when the network of canals (main canals, branch canals, distributaries etc.) are provided, then canals may have to cross the natural drainages like rivers, streams, drains valley and other canals at different points within the commanded area of the project. The crossing of the canals with such obstacles cannot be avoided. So, suitable structures must be constructed at the crossing point for the easy flow of water of the canal and drainage in the respective directions. These structures are known as cross drainage structures. They are generally very costly, and should be avoided if possible by changing the canal alignment and/or by diverting the drains. The nature of cross drainage works may be different at different places. Sometimes, the bed level of canal may be below the bed level of drainage and sometimes, it may be higher than that of the drainage. The bed level of canal and drainage may be nearly same also. So the structures are different at different places and the designations of the structures also are different. The details of these various structures will be dealt in this chapter.

8.1 Necessity of Cross – Drainage works

The following factors justify the necessity of cross drainage works:

- The watershed or ridge canal do not have cross drainages. But in actual orientation of the canal network, this ideal condition may not be available and the obstacles like natural drainages may be present across the canal. So, the cross drainage works must be provided for running the irrigation system.

- At the crossing point, the water of the canal and the drainage may get intermixed. So for the smooth running of the canal with its design discharge the cross drainage works are required.

- The site condition of the crossing point may be such that without any suitable structure, the water of the canal and drainage cannot be diverted to their natural directions. So the cross drainage works must be provided to maintain their natural direction of flow.

8.2 Types of Cross Drainage Works

Based on the relative bed levels, maximum water levels and relative discharges of the canals and drainages the cross drainage works may be of the following types for 3 cases.

Case I: Irrigation Canal Passes Over the Drainage

1. Aqueduct

An aqueduct is a cross-drainage structure constructed where the drainage flood level is below the bed of the canal. Small drains may be taken under the canal and banks by a concrete or masonry barrel (culvert), whereas in the case of stream crossings it may be economical to flume the canal over the stream. In this case, the drainage water passes clearly below the canal.

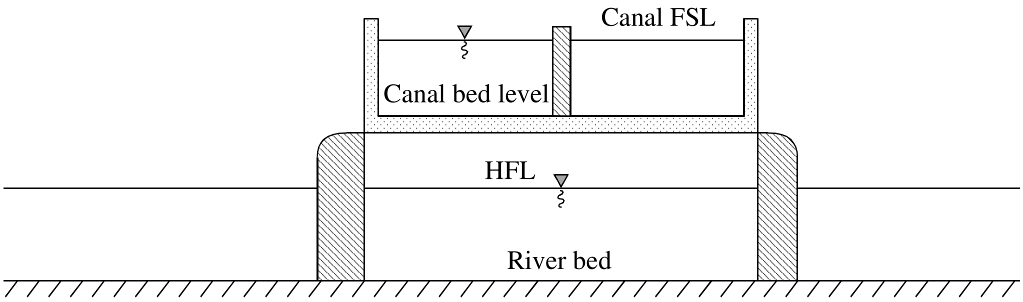


Figure 8.1: Section of aqueduct

2. Siphon Aqueduct

In a hydraulic structure where the canal is taken over the drainage, but the drainage water cannot pass below the canal freely. It flows under siphonic action and designed for total head loss through inverted siphon. So it is known as siphon aqueduct. This structure is suitable when the bed level of canal is below the highest flood level of the drainage.

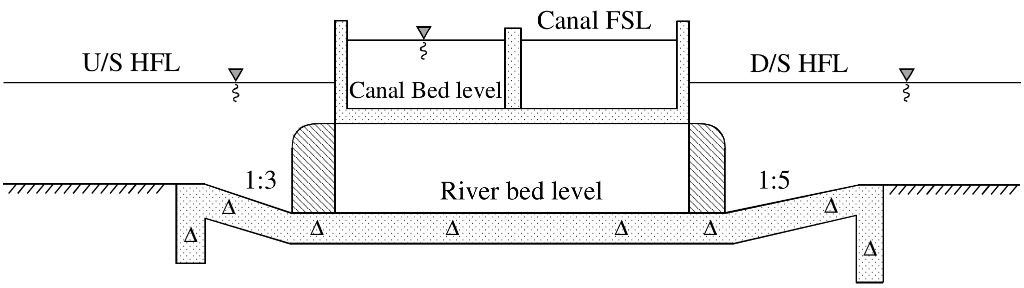


Figure 8.2: Section of siphon aqueduct

Case II: Drainage passes over the irrigation canal.

1. Super passage

The hydraulic structure in which the drainage is taken over the irrigation canal is known as super passage. The structure is suitable when the bed level of drainage is above the full supply level of the canal. The water of the canal passes clearly below the drainage.

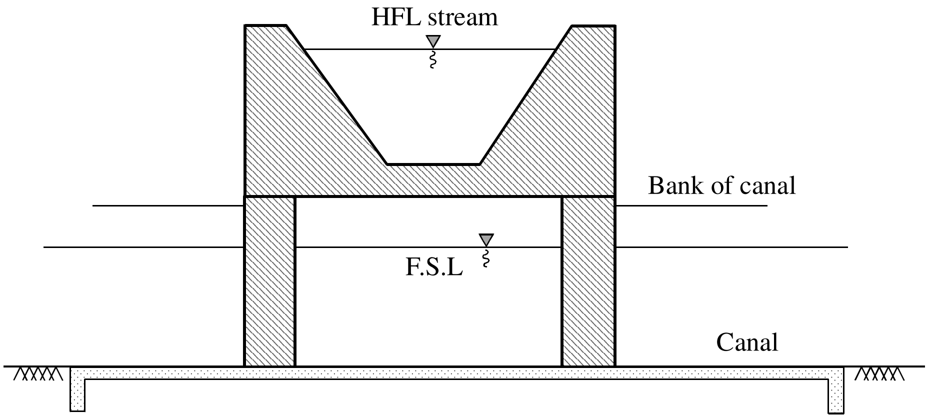


Figure 8.3: Super passage

2. **Siphon (Canal Siphon or Siphon Super Passage)**

   The hydraulic structure in which the drainage is taken over the irrigation canal, but the canal water passes below the drainage under siphonic action is known as canal siphon or siphon or siphon super passage. This structure is suitable when the bed level of drainage is below the full supply level of the canal.

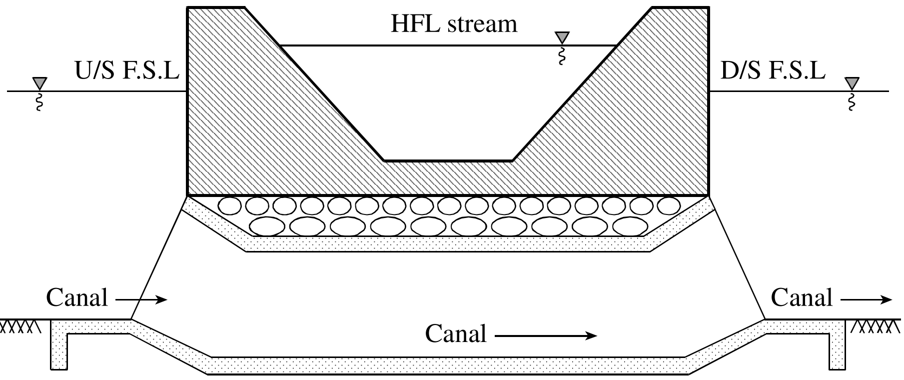


Figure 8.4: Siphon / siphon super passage /canal siphon

Case III: Drainage and canal intersection each other at the same level.

1. **Level crossing**

   Level crossing facilities are provided when both the drain and the canal run at more or less the same level. This is more frequently used if the drain flows for a short period (e.g. flash floods in the drain); in addition, the mixing of the drain flows to canal may be acceptable (if drain is siltless).

   The plan layout of a level crossing with two sets of regulators, one across the drain and the other across the canal, is shown in figure 8.5. Normally, the canal regulator regulates its flow with the drain regulator kept closed. Whenever the flash floods occur, the canal gates are closed and drainage gates opened to let the flood flow pass.

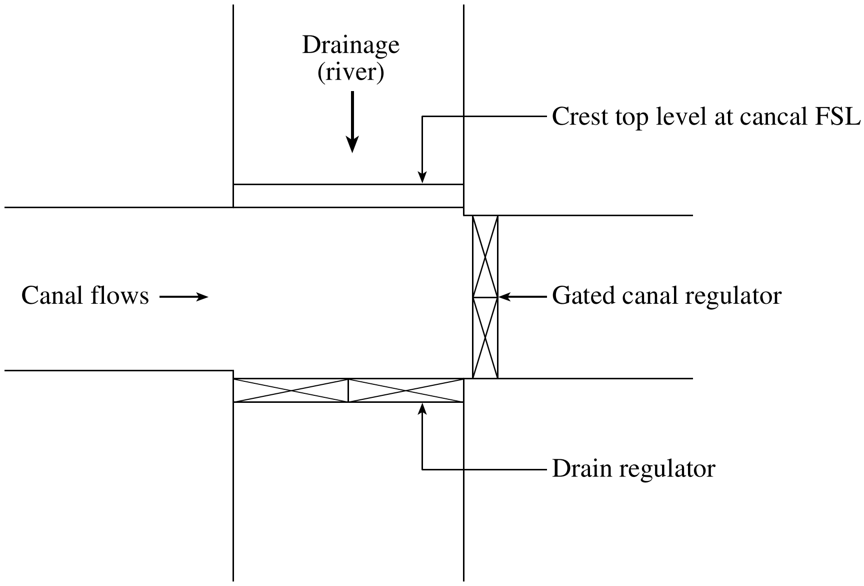


Figure 8.5: Level Crossing

2. Canal inlets and outlets

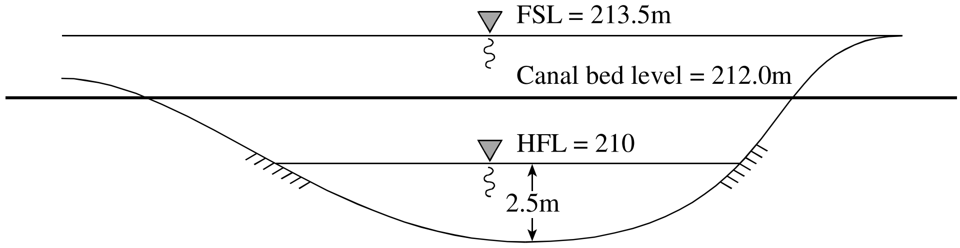
When the drainage flow is small it may be absorbed into the canal through inlets. The flow in the canal may be balanced, if necessary (in the case of small canals), by providing suitable outlets (or escapes). The inlet and outlet structures must also be provided with energy dissipators wherever necessary.

Example 8.1

From the following data, sketch the suitable type of cross drainage structure:

Canal	Drainage
Full supply discharge= 32 sumecs	High flood discharge = 300 cumecs
Full supply level =213.5 m	High flood level = 210.0m
Canal bed level = 212.0m	High flood depth = 2.5m
Canal bed width= 20m	General ground level= 212.5m

Solution:



From figure and given data; canal bed level is much above than HFL of drainage, than **aqueduct** is appropriate for cross drainage structure.

8.3 Design of Siphon Aqueduct

Depending upon the cross – section of the canal over the barrel or (culvert), the aqueducts or siphon aqueducts are classified into the following three types:

Type I Aqueduct

In this type of aqueduct (or siphon aqueducts) the cross – section of the canal is not changed. The original cross – section of the canal with normal side slopes is thus retained. The length of the barrel through which the drainage passes under the canal is a maximum in this type of structures, because the width of the canal section is maximum. In this type of structures the canal wings are not required. This type is suitable when the width of the drainage is small (say less than 2.5 m). If the section is changed, the cost of canal wings would be large in comparison to the saving resulting from decreasing the length of culvert.

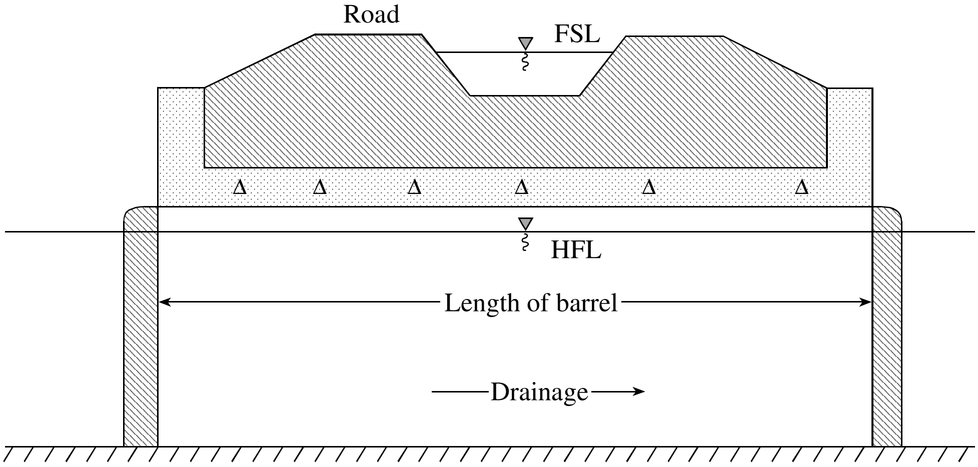


Figure 8.6: Type I Aqueduct

Type II aqueduct

In this type of aqueduct (or syphon aqueduct), the outer slopes of the canal banks are discontinued and replaced by retaining walls [Figure 8.7]. Thus the length of the barrel is reduced, but the cost of retaining wall is added to the overall cost. This type of structure is suitable when the width of the drainage is moderate (say 2.5 m to 15 m) so that the cost of retaining walls is less in comparison to the saving resulting from decreasing the length of drainage barrel.

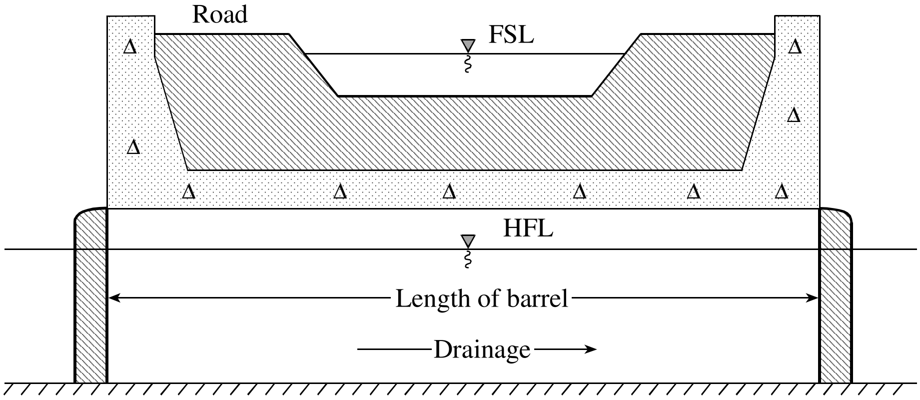


Figure 8.7: Type II Aqueduct

Type III aqueduct

In this type of aqueduct (or syphon aqueduct), the entire earth section of the canal is discontinued and replaced by a concrete or masonry through over the drainage [Figure. 8.8].This type of structure is generally suitable when the width of the drainage is very large (say more than 15 m), so that the cost of the trough and canal wing walls is less in comparison to the saving resulting from decreasing the length of barrel. In this type of structure, the canal can be easily flumed, which further reduces the length of the drainage barrel.

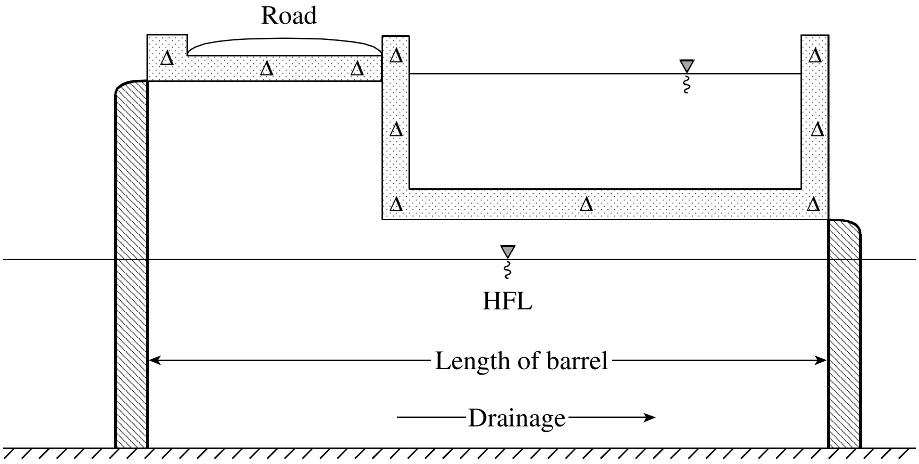


Figure 8.8: Type III Aqueduct

Selection of suitable type of aqueduct

The selection of the suitable type of canal cross-section depends upon the width and discharge of the drainage. A very small drainage requires a type I aqueduct, which in many cases may be merely a pipe or a small culvert passing under the canal. On the other hand, over a river of a large size type III aqueduct would be the most economical. For moderate size of the drainage, type II aqueduct may be most suitable.

However, the actual limits with regard to the size of drainage for which one particular type of aqueduct will be most suitable will vary with local conditions and the cost of construction. Comparative estimates should be prepared for finding out the most economical type of aqueduct for a particular site.

Design a Drainage water way

The waterway required to be provided for the drain is given by Lacey's equation for regime perimeter as $P = 4.75\sqrt{Q}$

Where P = wetted perimeter in m

Q = maximum flood discharge in cumecs.

In general, the total waterway between the abutments is made equal to Lacey's regime perimeter. For small drains a contraction up to 20% beyond Lacey's regime perimeter is also permissible. Since the piers will occupy a part of this total waterway, the clear water way will be less than Lacey's regime perimeter to the extent of total thickness of the piers. This is however, permissible because under the work the regime conditions do not exit. Moreover, the idea of relating the waterway to Lacey's regime perimeter is to develop a stable channel between training banks upstream of the work. In wide shallow rivers no distinction is made between the width of waterway and wetted perimeter but in small drains the width of water may be adjusted to provide the required perimeter.

Size of barrel:

After having fixed the waterway the size of barrels is to be fixed. In the case of an aqueduct, the canal trough is carried clear above the HFL and hence the height of openings is automatically fixed. In the case of a siphon aqueduct, the required area of waterway is obtained by dividing the fixed discharge by the permissible velocity of flow through the barrels. The velocity of floor through the barrels is normally limited to 2 to 3 m/sec. higher velocities may cause quick abrasion of the barrel surfaces by rolling grit and will result in high afflux upstream of the siphon aqueduct thus requiring higher and longer marginal banks to avoid the flooding of the country side. The total area divided by the number of spans gives the area required by each span. Knowing the width of span and the shape of the opening, the required depth of the opening can be determined which will fulfill the area required of the span. The span to be provided depends entirely on structural and economic considerations. For instance where foundations are costly fewer piers or longer spans are provided.

From the canal bed level, deducting the thickness of the culvert or barrel roof plus the depth of the opening, the level of the floor of the barrel is obtained. Further in the case of siphon aqueduct it is desirable to leave some clear headway between the bed of the drain on the downstream side of the crossing and underside of the culvert or barrel roof. This headway is desirable to minimize the blocking of culverts or barrel by silt rolling along the bed of the drain and it should be either 1m or half the height of culvert or barrel, whichever is less.

Design a Canal water way

In the case of siphon aqueduct the canal is flumed i.e. the canal waterway is contracted, to economize the cost of cross drainage works. The flumed portion or the trough is gradually connected to the normal section with smooth transition wings at both ends. The contraction of canal waterway although reduces the barrel length s or the width of the works but it involves the provision of extra transition wings. As such the canal waterway should be flumed to such an extent that the cost of the works as a whole is minimum.

The fluming or the contraction of the canal waterway should be done in such a way that the velocity in the trough is not more than 3 m/sec and the flow remains subcritical to avoid the possibility of hydraulic jump forming in the trough. The approach or contraction transition should not be steeper than 30^0 which corresponds to a splay of 2:1 and the departure or expansion transition should not be steeper than 22.5^0 which corresponds to a splay of 3:1. The transitions consist of curved and flared wing walls so that there is minimum loss of head and the flow is streamlined.

Design of transition

The transitions can be designed according to two different conditions as indicated below.

1. Design of transitions when the water depth remains constant.
2. Design of transitions when the water depth varies.

Design of transitions when the water depth remains constant:

In the case the water depth in the transition and the trough remain constant the following two methods may be adopted for the design of transitions.

(i) Mitra's hyperbolic transition

For water depth to remain constant in the transition and the trough, A.C. Mitra proposed hyperbolic transition which is given below.

$$B_x = \frac{B_c\, B_f\, L_f}{L_f\, B_c - (B_c - B_f)x}$$

Where, B_c and B_f are bed width of normal canal and flumed section respectively.

B_x be the bed width at a distance x from the flumed section.

L_f be the total length of the transition.

(ii) Chaturbedi's semi cubical parabolic transition.

R.S. Chaturvedi on the basis of his experiments proposed the following equation for semi – cubical parabolic transitions when water depth remains constant.

$$x = \frac{L_f B_c^{3/2}}{B_c^{3/2} - B_f^{3/2}} \left(\frac{B_f}{B_x}\right)^{\frac{3}{2}}$$

Choosing various convenient values of B_x, the corresponding distance x can be computed.

Design of flumed section

As shown in figure 8.10(a) the contraction transition starts at section 1 – 1 and ends at section 2 – 2. The flumed section continues from section 2 – 2 to section 3 – 3. The expansion transition starts from section 3 – 3 and ends at section 4 – 4. From section 4 – 4 the channel flows in its normal cross section and the conditions at this section are completely known. Let D and V with appropriate subscripts refer to depths and velocities at the various sections. The design is done in the following steps.

Step 1: Let the bed level and cross section of the canal at section 4 – 4 completely known.

Water surface elevation at 4 – 4 = Bed level at section 4 – 4 + D_4

TEL at section 4 – 4 = Water surface elevation at 4 – 4 + $\frac{V_4^2}{2g}$

Step 2: Between sections 3 – 3 and 4 – 4 there is energy loss due to expansion and due to friction. The energy loss due to expansion may be taken as $0.3\left(\frac{V_3^2 - V_4^2}{2g}\right)$. The energy loss due to friction is generally small and may be neglected.

Since trough dimensions at section 3 – 3 are known, V_3 is known.

Elevation of TEL at section 3 – 3 = Elevation at section 4 – 4 + $0.3\left(\frac{V_3^2 - V_4^2}{2g}\right)$

Water surface elevation at section 3 - 3 = Elevation of TEL at section 3-3 – $\frac{V_3^2}{2g}$

Bed level at section 3 – 3 = Water surface elevation at section 3 - 3 – D_3

Step 3. Between sections 2 – 2 and 3 – 3 the channel flows in a trough of constant cross – section and there is uniform flow. The only loss in the trough is therefore friction loss H_L which can be computed by Manning's equation as indicated below.

$$V = \frac{1}{n} R^{2/3} S^{1/2}$$

$$V = \frac{1}{n} R^{2/3} \left(\frac{H_L}{L}\right)^{\frac{1}{2}}$$

$$H_L = \frac{V^2 n^2 L}{R^{4/3}}$$

Elevation of TEL at section 2 – 2 = Elevation of TEL at section 3 – 3 + H_L

Water surface elevation at section 2 – 2 = Elevation of TEL at section 2 – 2 - $\frac{V_2^2}{2g}$

Bed level at section 2 – 2 = Water surface elevation at section 2 - 2 – D_2

Step 4: Between sections 1 -1 and 2 – 2 there is energy loss due to contraction as well as due to friction. The energy loss due to contraction may be taken as $0.2\left(\frac{V_2^2 - V_1^2}{2g}\right)$. The energy loss due to friction is generally small and may be neglected.

Elevation TEL at section 1 – 1 = Elevation of TEL at 2 – 2 + $0.2\left(\frac{V_2^2 - V_1^2}{2g}\right)$

Water surface elevation at section 1 – 1 = Elevation of TEL at section 1 – 1 - $\frac{V_1^2}{2g}$

Bed level at section 1 – 1 = Water surface elevation at section 1 – 1 – D_1

Head loss through siphon barrels

In the case of siphon aqueducts and siphon the head loss through the siphon barrels may be calculated by using Unwin's formula which is as follows.

$$h = \left(1 + f_1 + f_2\frac{L}{R}\right)\frac{V^2}{2g} - \frac{V_a^2}{2g} \qquad \ldots 8.1$$

Where h = head loss through the siphon barrel in m, which is equal to the difference between water levels upstream and downstream of siphon barrels

L = length of the barrel in m

R= hydraulic mean radius of the barrel in m

V= velocity of flow through the barrels in m/sec

V_a = velocity of approach in m/sec which is generally neglected

f_1 = coefficient for loss of head at entry, which may be taken as 0.505 for an unshaped mouth of the same cross – sectional area as the barrel and 0.08 for bell mouth and

f_2 = coefficient such that the loss of head in the barrel due to friction is given by $f_2\frac{LV^2}{2gR}$

The value of f_2 is given by $f_2 = a\left(1 + \frac{b}{R}\right)$ in which the values of a and b for different materials may be taken from the following table.

Nature of surface of barrel	a	b
Smooth iron pipe	0.00497	0.025
Encrusted pipe	0.00996	0.025
Smooth cement plaster	0.00316	0.030
Brick work	0.00401	0.070
Rubble masonry or stone pitching	0.00507	0.25

As stated earlier the velocity of flow through the siphon barrel is limited to 2 to 3 m/sec. Thus knowing the velocity of flow the head required to generate the same velocity can be found from equation 8.1. The high flood level of the drain downstream of the work remains unchanged even after the construction of the work, but on the upstream side the high flood level increase by the amount of afflux or the total head loss as given by equation 8.1.

Uplift pressure on the underside of the trough (on the barrel roof)

When water is flowing through the siphon barrel and the water level on the downstream side at the barrel is higher than the underside of the trough (or the barrel roof), uplift pressure is exerted on the underside of the trough (or the barrel roof). At the downstream end of the barrel the uplift pressure on the underside of the trough is equal to the difference between the downstream water level and the level of the underside to the trough. At any other point along the barrel, the uplift pressure is given by the ordinate between the hydraulic gradient line and the underside of the trough. The afflux having been calculated by Unwin's formula the hydraulic gradient line can be drawn as shown in figure below. The maximum uplift pressure occurs on the underside of the trough at the upstream end of the barrel. The uplift pressure would be maximum when the highest flood is passing through the barrel and there is no water in the trough at that time. Another critical condition to be considered for the design of the trough bed (or barrel roof) is when the trough is running full with dead weight of trough but there is no uplift from the barrel.

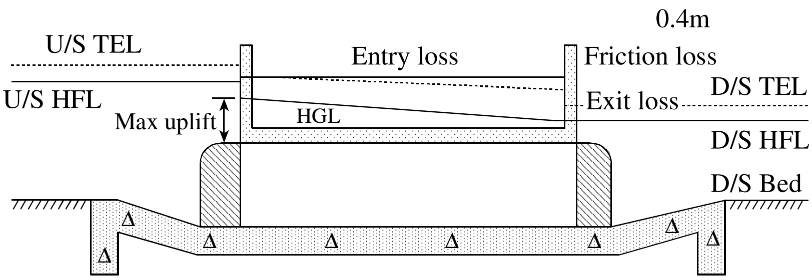


Figure 8.9: Uplift pressure on the underside of the trough

The thickness of the slab forming the trough bed (or the barrel roof) required to counter balance the uplift (i.e. for the first condition noted above) is generally less than what is required to support the load of trough water (i.e. for the second condition noted above). However, in some case the thickness of the slab required to counterbalance the uplift may exceeds the thickness required to support the water load. In each of these two cases if the slab is designed to balance the uplift or the water load by gravity only i.e. providing thick slab, which is usually undesirable because it leads to lowering of the levels of both the underside of the trough bed (or the barrel roof) and the barrel floor which in turn results in increasing the uplift on the trough bed as well as on the barrel floor.

As such one of the alternatives would be to provide a reinforced concrete slab with reinforcement at the bottom to take the water load of the trough and the reinforcement at the top to resist uplift by bending. It may however be noted that since a part of the uplift is resisted by gravity (i.e. by the self-weight of the slab) only the remainder of the uplift after deducting the part counter balanced by gravity is to be resisted by bending. Hence the reinforcement at the top of the slab is to be provided only for the remainder of the uplift. Another alternative may be to provide the thickness of the slab to support the water load in the trough by gravity and to provide reinforcement only at the top of the slab to resist the remainder of the uplift by bending. In both these cases it would, however, be necessary to anchor the slab through the pier to the bottom floor by steel bolts to provide the necessary end reactions for upward bending. A better alternative may be to provide R.C.C. box culvert.

Example 8.2:

Design a siphon aqueduct for the following data:

	Canal	Stream
Discharge (m^3/sec)	30	500
Bed level	200.00	198.00
Canal FSL	202.00	
Bed width	25.00	
Canal side slopes	1.5:1 (H:V)	
Stream HFL		200.50

The general terrain level is 200 m.

Solution:

Here; canal bed level 200.00 m is slightly below the HFL (200.50 m) and the HFL is below the FSL of canal (202.00 m), hence a siphon aqueduct is required.

Step 1: Design of drainage water way:

Lacey's Regime perimeter $P = 4.75\sqrt{Q}$

$$P = 4.75\sqrt{500} = 106\text{m}.$$

Provide 12 piers of 1.25m thickness; we have 13 span of 7m each.

The length occupied by 13 bays of 7m each = 91

Length occupied by 12 piers of 1.25m each = 15m

The total length of water way = 91 + 15 = 106m which is satisfactory.

Assume maximum velocity through the siphon barrels = 2m/sec.

Height of siphon barrel required $= \dfrac{Q}{\text{clear waterway} \times \text{velocity}} = \dfrac{500}{13 \times 7 \times 2} = 2.747\text{m}$

Provide rectangular barrels of 7 m wide and 2.75m high.

Step 2: Design of canal waterway

Normal bed width of canal = 25 m

Since the drainage width is large (106m at the crossing) it is economical to flume (concrete, n = 0.014) the canal.

Adopt a maximum flume ratio of 0.5. Therefore the flumed width of canal (trough) = 0.5 × 25 = 12.5m.

Provide a splay of 2:1 in contraction and a splay of 3:1 in expansion. (Hinds, 1928)

Length of transition in contraction = $\dfrac{(25 - 12.5) \times 2}{2} = 12.5\text{m}$

Length of transition in expansion = $\dfrac{(25 - 12.5) \times 3}{2} = 18.75\text{m}$

The length of trough from abutment to abutment = 106m

Step 3: Design of flumed section with transitions

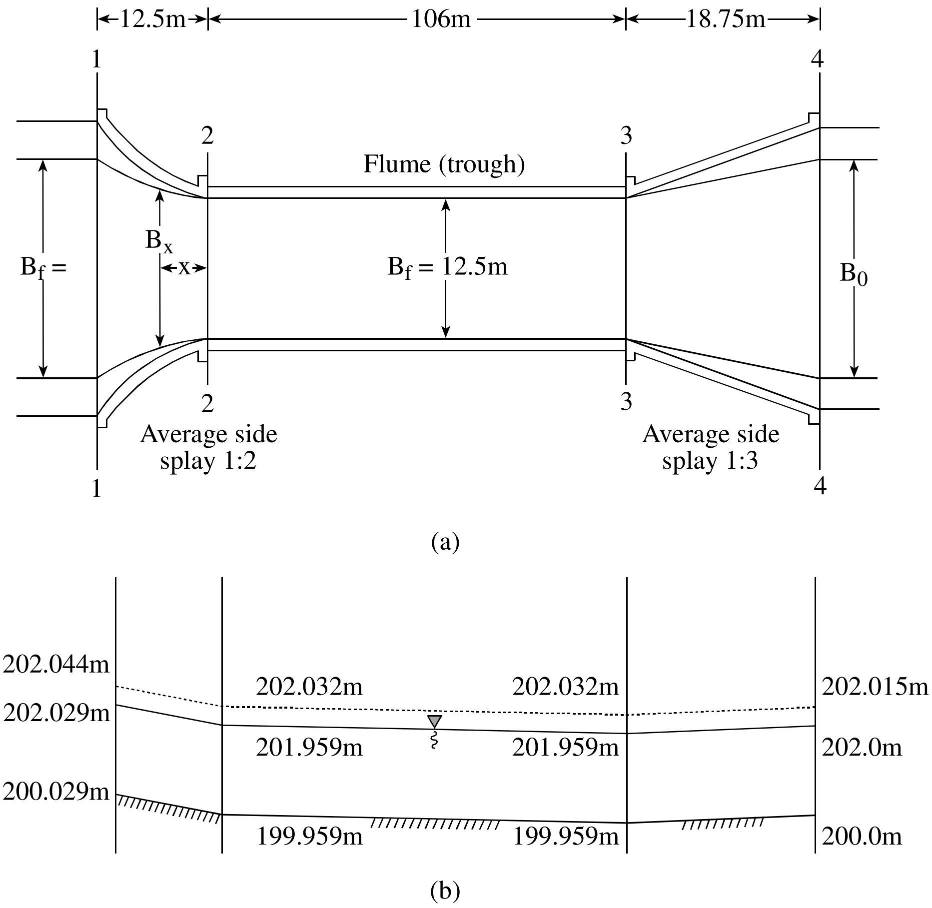


Figure 8.10: Flumed section with transition at different section

Referring to Figure 8.10 (a), the following results can be obtained to maintain a constant flow depth of 2.0 m (full supply depth). The calculations are achieved from section 4 – 4 and proceed towards section 1 – 1 as tabulated below.

	Section			
	4 – 4	**3 – 3**	**2 – 2**	**1 – 1**
Width (m)	25.00	12.5	12.50	25.0
Area of flow (m^2) ($By + zy^2$)	56.00	25.00	25.00	56.00
Velocity (m/sec) Q/A	0.536	1.2	1.2	0.536
Velocity head (m) $V^2/2g$	0.015	0.073	0.073	0.015
Loss (m)		Expansion loss = 0.017	*Friction loss = 0.0004	Contraction loss = 0.012
R L of canal bed (m)	200.00	201.959 – 2 = 199.959	201.959 – 2 = 199.959	202.029 – 2 = 200.029
Flow depth (m)	2	2	2	2
RL of water surface (m)	200.00 + 2= 202.00	202.032 – 0.073 = 201.959	202.032 – 0.073 = 201.959	202.044 – 0.015 = 202.029
RL of TEL (m)	202.00 + 0.015 = 202.015	202.015+0.017 = 202.032	202.032 + 0.0004 = 202.032	202.032 + 0.012 = 202.044

***Friction loss is calculated below**

$$H_L = \frac{V^2n^2L}{R^{4/3}} = \frac{1.2^2 \times 0.016^2 \times 106}{29^{4/3}} = 0.0004$$

$$R = \frac{A}{P} = \frac{2 \times 12.5}{2 \times (2 + 12.5)} = 29$$

Step 4: Design of Transition

For a constant depth of flow the transition may be designed such that the rate of change of velocity per meter length of transition is constant. This approach yields the bed width of the transition at a distance x from the flume section as

$$B_x = \frac{B_0 B_f L_f}{L_f\, B_0 - x\,(B_0 - B_f)} \quad \text{Mitra's hyperbolic equation .... 8.2}$$

Where B_0 = Bed width at original section = 25m

B_f = Bed width of flumed channel section = 12.5m

L_f = length of transition = 12.5m for contraction and 18.75 for expansion transition.

B_x= bed width at any distance x from the flumed section.

For Contraction transition:

$$B_x = \frac{3906.25}{312.5 - x\,(25 - 12.5)}$$

X	0	2	4	6	8	10	12.5
B_x	12.5	13.586	14.88	16.44	18.38	20.833	25

For expansion transition:

$$B_x = \frac{5859.375}{468.75 - x\,(25 - 12.5)}$$

X	0	4	8	12	16	18.75
Bx	12.5	13.99	15.88	18.38	21.80	25

Step 5: Design of Trough

The trough shall be divided into two equal compartments of 6.25m wide, separated by 0.3 m thick partition. The entire trough (flume section) can be designed as a monolithic concrete structure. Provide side wall and a bottom slab of about 0.4m (to be fixed by the usual structural design methods).

Overall length of barrel = 12.5 + 0.3 + 2 × 0.40 = 13.60m.

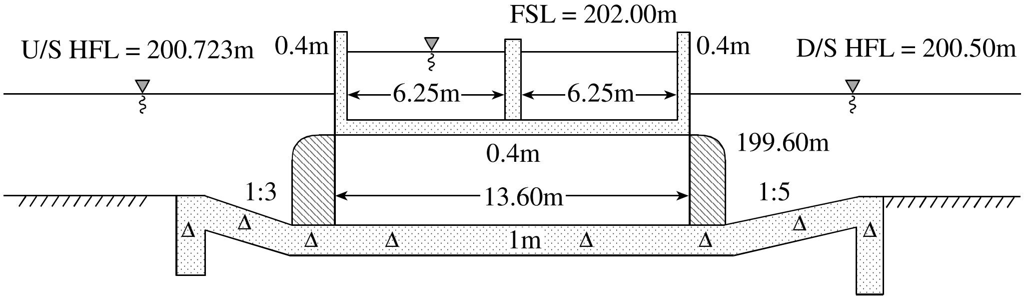


Figure 8.11: Longitudinal section of siphon aqueduct

Step 6: Head loss through the siphon barrels

The head loss through the siphon barrel is given by Unwin's formula as equal to

$$h = \left(1 + f_1 + f_2\frac{L}{R}\right)\frac{V^2}{2g}$$

Where V = velocity through barrels = 2 m/sec

f_1 = entry loss coefficient = 0.505 for unshaped mouth

f_2= a (1 + b/R) where the value of a and b for cement plastered barrels

a = 0.00316 and b = 0.030

$$R = \text{hydraulic mean depth for barrel} = A/P = \frac{7 \times 2.75}{2 \times (7 + 2.75)} = 0.9871\text{m}$$

L = length of barrel = 13.60

Now, $f_2 = 0.00316\,(1 + 0.030/0.9871) = 0.00326$

$$h = \left(1 + 0.050 + 0.00326 \times \frac{13.60}{0.9871}\right)\frac{2^2}{2g} = 0.223\text{m}$$

High flood level of drainage is given = 200.50m

Afflux h = 0.223m

U/S HFL = D/S HFL + h = 200.723m

Step 7: Uplift pressure on roof barrels

RL of bottom of trough = RL of canal bed – slab thickness = 200 – 0.40 = 199.60m

The entry loss at the barrel = $0.5\dfrac{V^2}{2g} = 0.102$m

Therefore the pressure head inside the barrel just downstream of its entry = 200.723 – 0.102 – 199.60= 1.021m of water × 9.81 = 10.01 KN/m^2.

RCC Slab should be designed by providing bottom reinforcement for dead weight of trough slab + 2 m of water load and by providing top reinforcement for uplift of 10.01 KN/m^2 minus dead weight of trough slab.

Exercise

1. The cross drainage structure across an irrigation channel has following data:
   i. Discharge of canal = 30 cumecs
   ii. Bed width of canal =20m
   iii. Full supply depth of canal =1.6m
   iv. Bed level of the canal =260.00m
   v. Side slope of canal =1.5:1 (H:V)
   vi. High flood discharge of drainage = 450 cumecs
   vii. High flood level of drainage =261.00 m
   viii. Bed level of drainage =258.00m
   ix. General ground level =260.00m

   Design the drainage waterway, canal waterway and find the bed levels and FSL at four different sections of the canal Trough
2. Following data are obtained at the crossing of a canal and a drainage:

	Canal	Stream
Discharge (m^3/sec)	20	200
Bed level	150.0	148.5
Canal FSL	151.5	
Bed width	12.00	
Canal side slopes	1.5:1 (H:V)	
Stream HFL		150.7

   The general terrain level is150.0 m.

   Design the following components of siphon aqueduct.
   i. Drainage waterway ii. Canal waterway
   iii. Transition iv. Uplift
3. Explain with sketch aqueduct, siphon aqueduct, siphon and super passage.
4. What is level crossing? Write down the suitability of inlet and out structure in canal irrigation system.
5. Which cross drainage structure is suitable and appropriate in Hills of Nepal?

✸✸✸

CHAPTER

9 Water Logging and Drainage

9.1 Water Logging

Water logging refers to the saturation of soil with water. Soil may be regarded as water logged when it is nearly saturated with water much of the time such that its air phase is restricted and anaerobic conditions prevails. In extreme cases of prolonged waterlogging, anaerobiosis occurs, the roots of mesophytes suffer, and chemical reduction processes occur including denitrification, methanogenesis and the reduction of iron and manganese oxides.

In agriculture, various crops need air (specifically oxygen) to a greater or lesser depth in the soil. Waterlogging of the soil stops air getting in. How near the water table must be the surface for the ground to be classed as water logged, varies with the purpose in view. A crop's demand for freedom for water logging may vary between seasons of the year, as with the growing of rice.

In irrigated agricultural land, waterlogging is often accompanied by soil salinity as waterlogged soils prevent leaching of the salts imported by the irrigation water. From the gardening pint of view, waterlogging is the process whereby the soil blocks off all water and is so hard it stones air getting in and it stops oxygen from getting in. (Hillel, 2004)

An agricultural land is said to be waterlogged when the soil pores within the root zone of the crops are saturated to such an extent that normal circulation of year with in the soil pores is totally cutoff. The water logging affects the productivity or fertility of the land and thus leads to reduction in the crop yield.

9.2 Causes, Effects and Preventive Measures of Water Logging

9.2.1 Causes of water logging

Basically, water logging is the rise of water table unto root zone level leading to carious problem to crop growth. This may occur due to the following reasons.

1. **Over and intensive irrigation**

   Under the practice of intensive irrigation, the maximum irrigable area of a small region is irrigated. This leads to, too much of irrigation, in that region, resulting in heavy percolation and subsequent rise of water table. 'This can be overcome by a policy of extensive irrigation, i.e. irrigation spread over wider regions. Thus, to avoid water logging extensive irrigation should supersede the policy of intensive irrigation.

2. **Seepage of water from nearby areas**

   Water from the adjoining high lands may seep into the sub-soil of the affected land and may raise the water table. This occurs generally when land surface is not flat and adjoining area is moisture rich.

3. **Seepage of water through the canals/ Reservoirs**

   This is major cause of water logging in canal commanded areas. This has reached an arming stage in many areas. Water may seep through the beds and sides of the adjoining canals, reservoirs, situated at a higher level than the affected land. This results into high water table in the affected area. This seepage is many times excessive particularly when soil at the site of canals and reservoirs is very pervious.

4. **Encounter of impervious obstruction**

We know that water seeping below the soil moves horizontally or laterally. This may encounter an impervious obstruction, causing the rise of water table on the upstream side of the obstruction. This may lead to water logging. Similarly, in certain cases it is possible that an impervious stratum may occur below the top layers of pervious soils. In this case also, water seeping through the pervious soils will not be able to go deep, and hence, quickly results in high water table rise.

5. **Lack of natural drainage system**

If sufficient availability of natural drainage is not there in form of slope, soils having less permeable sub-stratum such as clay lying below the top layers of pervious soils, will not be able to drain the water deep into the ground. This may lead to rise in water level to the extent that it can affect the root zone and the crop cultivation.

6. **Inadequate surface drainage**

Surface drainage system is common passage way for runoff water. It becomes necessary to ensure that storm water falling over the land and the excess irrigation water should be removed from the area. It should not be allowed to percolate below. In absence of proper surface drainage, the water will constantly percolate and will raise the level of the water table leading to water logging.

7. **Excessively high rain fall**

This is common source water logging is cities. Even in farm land heavy down pouring may cause water logging. However, excessive rainfall may create temporary water logging, but in the absence of good drainage, it may lead to continued water logging over the area

8. **Overgrowth of weeds and aquatic plant**

During rainy seasons weeds and grasses grow excessively obstructing the passage of water in natural waterways. If a land is continuously submerged by floods, aquatic plants like hyacinths, grasses and weed may grow. They may obstruct the natural surface drainage of the soil, and thus, increasing the chances of water logging.

9. **Irregular or flat topography**

Topography also affects natural drainage and thus leads to water logging. In steep terrain, the water is drained out quickly. On flat or irregular terrain having depressions, the drainage is very poor. These factors lead to greater detention of water on the land, causing more percolation and water logging, if infiltration of soil is not proper.

9.2.2 Effect of water logging

1. **Hampering the nitrification**

The life of a plant, in fact, depends upon the nutrients like nitrates, and the form in which the nitrates are consumed by the plants is produces by the bacteria, under a process called nitrification. These bacteria need oxygen for their survival. The supply of oxygen gets cutoff when the land becomes ill aerated, resulting in the death of these bacteria, and fall in the production of plant’s food (i.e. nitrates) and consequent reduction in the plant growth, which reduces the crop yield. Apart from ill aeration of the plants, water logging creates many other problems.

2. Delayed Cultivation practice

The normal cultivation operations, such as tilling, ploughing, etc. cannot be easily carried out in wet soils. In extreme cases, the free water may rise above the surface of the land, making the cultivation operations impossible. In ordinary language, such a land is called a swampy land. In general, this leads to excessive delay in cultural practices and delayed sowing of crop, less or very poor yield.

3. Overgrowth of weeds

Certain water loving plants like grasses, weeds etc. grow profusely and luxuriantly in waterlogged lands, thus affecting and interfering with the growth of the crops.

4. Rise of salt in surface layer

In a waterlogged land continuous upward flow of water is established from the water table to the ground surface. This is so because from the water table water rises into the capillary zone from where it is lost through evaporation and transpiration and more water is taken in form the water table to replenish these losses. As the water rises up it brings with it dissolved salts such as sodium carbonate, sodium chloride and sodium sulphate from the underlying layers to the surface and as water evaporates it leaves behind a deposit of these salts on the surface. The process of salt deposition in this manner is called salt efflorescence. Excessive concentration of salt in root zone of plant does not allow the plants to thrive which results in reduction in crop yield. Moreover with a heavy concentration of salt the soil may become alkaline which is highly unproductive and difficult to reclaim.

5. Restricted root growth

If the water table is high the roots of the plants are confined to the top layers of the soil above the water table and hence their growth is restricted. On the other hand if the water table is low the roots of the plants have more space for their growth. The restricted root growth will result in reduced crop yield.

6. Lower soil temperature

Well drained soils warm up faster than saturated soils. The low soil temperature which results for excessive moisture in the soil hampers the germination of seed, restricts development of root system and affects the rate of ripening. Moreover at low temperature of soil, the activity of the bacteria becomes sluggish and consequently less food is available for the plants. This results in lowering the crop yield.

7. Plant disease

When low soil temperatures are accompanied by poor aeration the root system of plants does not develop properly and becomes vulnerable to attack by pests and diseases. The healthy growth of plants is thus hampered resulting in reduced crop yield.

9.2.3 Preventive measures of water logging

It is evident that water logging can be controlled only if the passage and quantity of water into the sub soil below the surface is controlled and reduced. For this first of all, the inflow of water into the underground reservoir should be reduced and the outflow from this reservoir should be increased simultaneously. It is necessary to keep the highest position of water-table at least about 3 m below the ground surface. The various measures adopted for controlling water logging are given below.

1. **Lining of canals and water courses**

   Control measures must try to check the causes of water logging. As seepage is one of the main culprits of rise of water table, attempts should be made to reduce the seepage of water from the canals and watercourses. This can be achieved by lining them. It is a very effective method of controlling water logging.

2. **Reduced intensity of irrigation**

   The areas where there is a possibility of water logging, intensity of irrigation should be reduced. Efficient water supply should be ensured. Only a specific portion of irrigable land should receive canal water in one particular season. The remaining areas can receive water in the next season, by rotation. This can be achieved by crop rotation. This would help controlling water logging in the region.

3. **Crop rotation**

   Crop rotation is also an effective means of controlling water supply to sub surface zone. As certain crops require more water and others require less water. If a field is always sown with a crop requiring more water, the chances of water logging are more. In order to avoid this, a high water requiring crop should be followed by one requiring less water. We may choose next crop one requiring almost no water. For example, rice may be followed by wheat. And a dry crop such as cotton may follow wheat. This will ensure break in continuous excess irrigation.

4. **Optimum use of water**

   Excess of anything is harmful we know that only a certain fixed amount of irrigation water is required for best productivity. Any major deviation reduces the yield and empties the pocket of the farmer. But there is ignorance in some areas that using more water can increase crop yield. This happens more in the areas where water charges are less or where there is uncertain supply of canal water and electricity in case of tube well irrigation. Educating the farmers by proper extension method can bring improvement. As a policy matter, the revenue should not be charged on the basis of irrigated area but should be charged on the basis of the quantity of water utilized.

5. **Intercepting drains**

   These drains are to check the canal water seepage. Intercepting drains along the canals should be constructed, wherever necessary. They would help intercepting seepage water and prevent the water from reaching the area and thus water logging may be prevented.

6. **Improved natural drainage**

   The worn out natural drainage systems in the cropped area should be revived. This would reduce the percolation by, not allowing water to stand for a longer period. At community level, some relief in this direction can be obtained by removing the obstructions from the path of natural flow. This can be achieved by removing bushes and other obstructions and improving the slopes of the natural drainage lines.

7. **Efficient drainage system**

   An efficient drainage system should be provided in order to drain away the storm water and the excess irrigation water. A good drainage system consists of surface drains as well as subsurface drains.

8. Conjunctive use of surface and subsurface water

Conjunctive use is a combined use of sub-surface water or ground water and the surface water or canal water in a judicious manner to derive maximum benefits. The introduction of lift irrigation to utilize ground water helps in lowering the water table in a canal irrigated area, where water table tends to go up. This system ensures use of the ground water in conjunction with canal water for irrigation. The continuous use of ground water will not allow any appreciable rise in the level of water table, even due to continuous seepage of canal water. Thus, conjunctive use should be adopted to control water logging.

9.3 Drainage

Irrigation and drainage are two face of the same coin. Surface irrigation is a boon only if it is practiced with great care. As stated earlier, only optimum amount of water should be applied to the crop, as per the requirement of the crop and the properties of the soil. In fact, the root zone of the soil fails to absorb excess water which may percolate and help in raising the water table. If this gravity water encounter an impervious stratum and is not drained up properly, then this excess water is harmful to crop yield. It becomes necessary to remove excess water by draining it out from below the soil. The drained water may be discharged back either into a river or a canal or some other safe place. Hence, while designing a canal irrigation network, it is desirable to provide a suitable drainage system, for removing the excess irrigation water. Thus irrigation and drainage go together. Drainage system is also required for draining out the rain water and ensures its easy disposal to prevent its percolation.

Drainage system can be classified as:

a. Surface drainage b. Sub – surface drainage

Surface drainage is also called open drainage system while sub – surface drainage as tile – drainage or underground drainage.

9.4 Surface Drainage System and their Design

Surface drainage is the removal of excess rainwater falling on the fields or the excess irrigation water applied to the fields, by constructing open ditches, field drains, and other related structures. In this process the land is sloped towards these ditches or drains, as to make the excess water flow in to these drains. In fact, land grading, which results in a continuous land slope towards the field drains, is an important part of a surface drainage system. Land grading or land leveling is also necessary for surface irrigation

Types of surface drain

Surface drains are needed for removing the storm water and excess farm water, for most of the cultivated crops on .flat or undulating topography However, if no impervious layer occurs below the farm land and the water table is sufficiently lower, internal soil drainage are sufficient and no additional drainage facility is required. But for maximum productivity of most of the crops drainage facility becomes essential, particularly in waterlogged areas. Surface drains are two types shallow and deep surface drains.

1. Shallow surface drains

The open drains, which are constructed to remove the excess irrigation water collected in the depressions on the fields, as well as the storm (rain) water, are broad and shallow, and are called shallow surface drains. These drains carry the runoff to the outlet drains. They are trapezoidal in cross-section. If designed properly, they should carry the normal storm water from the fields, plus the excess irrigation water. Many a times, the excess irrigation water is neglected and these drains are designed only for the runoff resulting from the average storms which is neither economical nor desirable. Manning's equations may be used to obtain design velocity of these drains, keeping the velocity within the critical velocity, and thereby avoiding silting or scouring. Proper shape is selected based on available information. Manning's equation may however be used for the design of shallow as well as deep surface drains.

2. Deep surface drains

The drains, which are large enough to carry the flood water of the catchment area from the shallow surface drains, and are of sufficient depths to provide outlets even for the underground tile drains, if provided are called deep surface drains. These drains carry the storm water discharge, drains, shallow surface drains, and the seepage water coming from the underground tile drains. They are, therefore, designed for the combined discharge of the shallow surface drains as well as that of the tile drains. Sub-surface drains are required for soils with poor internal drainage and a high water table generally, a cunnette of about 0.6 m depth with steeper slope is provided in the center of the drain bed, so as to carry the seepage water of the underground tile drains. Cunnette is lined so as to withstand higher flow velocities. The full section of drain is used only during the rainy season when the cunnette is not able to handle the flow will Manning's equation may be used for estimating velocity and based on that flow capacity can be determined while designing of deep surface drains.

Different forms of sub-surface drains are described below.

1. Surface inlet

A surface inlet is intake structure constructed to carry the pit water into the sub-surface or tile drain. A cast iron pipe or a manhole constructed of brick or monolithic concrete, is sufficient and satisfactory. Basically, it is the facility to remove the surface water from the pot holes depressions, road ditches, and farmstead. This may also be accomplished by connecting them with the shallow surface drains called random field drains

2. French drain

When the quantity of water to be removed from the pits or depression is small, a blind inlet may be installed over the tile drain which is also called French drain. These are constructed by back filling the trench of the tile drain with graded materials, such as gravel and coarse sand, or with corn cobs, straw and similar substances, as shown in Figure 9.1.

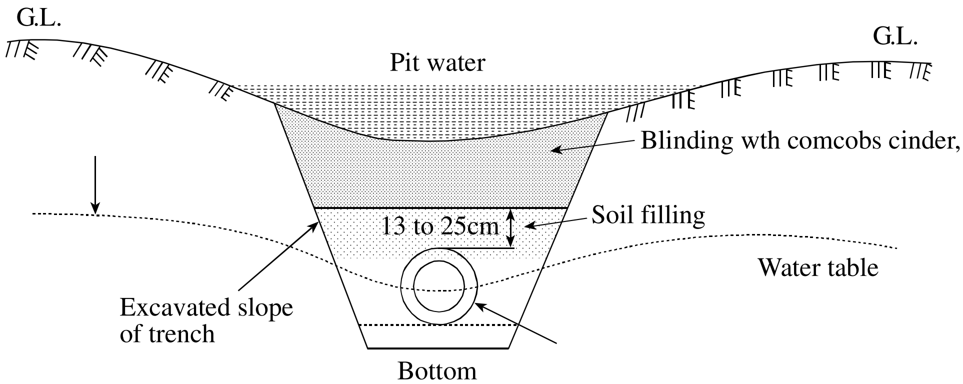


Figure 9.1: French drain

Such inlets are not permanently effective. The voids in the backfill of the blind inlet become filled up with the passage of time, thereby reducing its effectiveness. Even though they are not permanently effective, they are economical to be installed and do not interfere with the farming operations.

3. Bedding

Bedding is a method of surface which makes use of dead furrows, as shown in Figure 9.2. The area between the two adjacent furrows is known as a bed. The depth of the bed depends on the soil characteristics and tillage practices. In the bedded area, the direction of fanning may be parallel or normal to dead furrows. Tillage practices, parallel to the beds, retard water movement to the dead furrows.

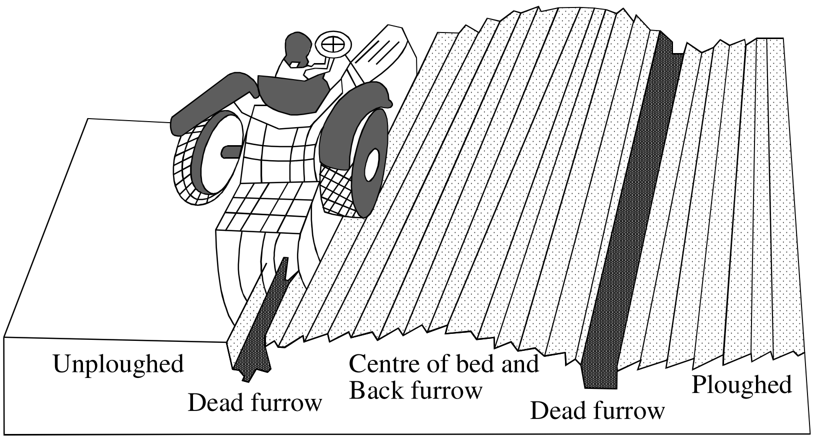


Figure 9.2: Bedding

Ploughing is always parallel to the dead furrows. Bedding is most practicable on flat slopes of less than 15%, where the soils are slowly permeable and the drainage is not economical.

9.4.1 Layout planning of drainage

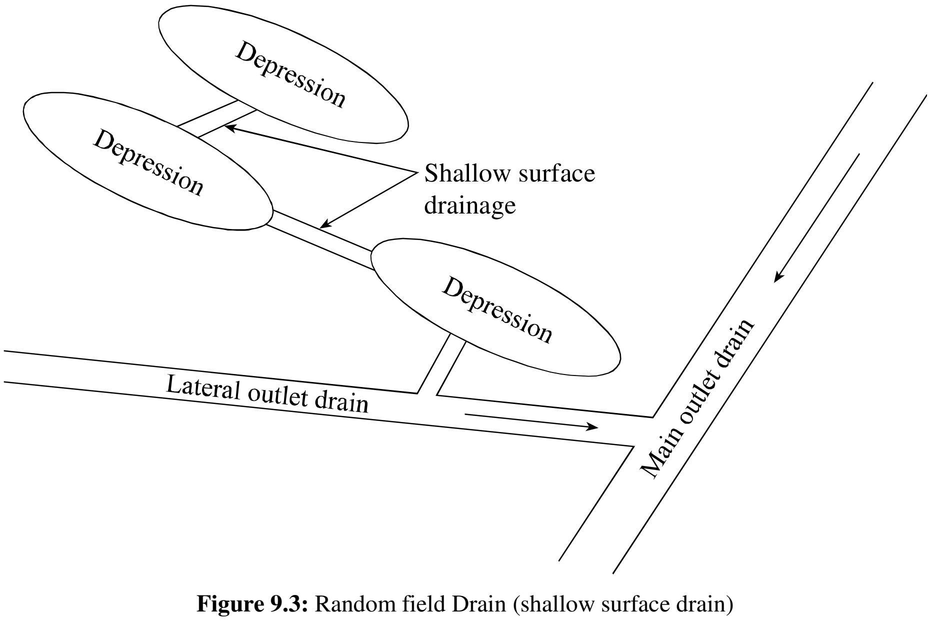


Figure 9.3: Random field Drain (shallow surface drain)

9.5 Drain design discharge

Strom drainage, in relation to irrigation schemes, falls into two main categories. Eternal drainage refers to runoff outside the irrigated area and is considered in the design of cross – drainage structures and headwork. Internal drainage refers to the runoff of excess water from the irrigated area itself. The runoff from these fields is characterized by the reduction of flow due to the storage capacity of the bunds. Internal drainage can be as a result of runoff from intense rainstorms or removal of an excess if irrigation water on the field, as at harvesting time. The critical events for the design of the drain capacities are the severe rainstorms.

In Nepal the predominant crop in the wet season is paddy rice; dry root crops, such as wheat and pulses, being grown in the winter/dry season when rainfall is negligible. As rice can grow in saturated soil, in internal drainage system is require only to remove excess rainfall and irrigation water, and no drainage of the root zone is necessary. Cultivation practice have also developed for saturated soils and access for machinery is not required.

Most areas of Nepal have an existing natural drainage system. However, extreme storms in the summer can cause some areas in the Terai toe be inundated for many days causing the rice crop to be submerged, and in consequence, respiration by the crop in restricted and if flooded for a sufficient period, the crop to be submerged, and in consequence, respiration by the crop is restricted and if flooded for a sufficient period, the crop dies. In the hills, the natural drainage system is generally sufficiently well developed to prevent such inundation and consequently internal drainage of bunded fields in the hills is normally not necessary.

9.5.1 Internal drainage of Bunded fields

In Nepal, a broad distinction must be drawn between the physical details of drainage from the bunded field in the hills and those in the lowlands or Terai. The hills fields are normally bunded bench terrace. The bund heights are typically 100mm. The spillage is rapid and unimpeded. The Terai fields are generally larger with typical bund heights of 200 mm. The topography is flat so that drainage is relatively slow, and widespread flooding may occur.

Terai

The runoff from bunded fields e.g. rice fields, is estimated from a simple water balance. This approach has been used in Nepal at the Narayani and Sunsari Morang irrigation projects.

The water balance incorporates the following assumptions:

- Initial water level is 40 mm
- Maximum water level is 300 mm which may persist for up to one day
- Depth in excess of 200 mm may persist for up to 3 days
- No rain follows the design rainfall for several days
- Losses due to evapotranspiration and deep percolation are replaced by ongoing irrigation and/or flood inflows during the design rainfall.

The balance may be expressed:

$$h = 40 + \frac{P}{3}t - Q \times t \qquad t \leq 3 \text{ days}$$

or, $$h = 40 + P - Q \times t \qquad t > 3 \text{ days.}$$

Where h = the depth of water in the field in mm

P = the design three day rainfall in mm

3 = days

t = the number of days that have elapsed since the rainfall began

Q = the drainage runoff in mm/day.

The 3 day rainfall is recommended for the design of field drains in the Terai. Where practicable, the return period adopted for design would be assessed from an analysis of the costs and benefits of drainage provision to particular standards. In the absence of such analysis, a 10 year return period is suggested as a guideline, and this has been used in the example 9.1.

For a given elevation and mean annual rainfall at the prospective site, the mean annual maximum 3 day rainfall should be obtain. To estimate the value with a 10 year return period, the mean maximum should be multiplied by a growth factor.

Growth factor $G = 0.32 \ln T + 0.78$

Where G = Growth factor

T= return period

Ln= natural logarithm

For 10 year return period Growth factor G = 1.5.

Example 9.1:

Design a surface drainage for a field of 40 ha area in Terai with following data. Design maximum yearly precipitation for three consecutive days = 500mm, longitudinal slope of channel 1:400, manning roughness coefficient 0.025, Maximum water level is 300mm which may persist for up to one day and depends in excess of 200 mm may persist for up to 3 days. Assume other suitable values if necessary.

Solution:

Here given that, design maximum yearly rainfall for 3 days (P) = 500mm. Hence no need to multiply by growth factor. The water balance h in mm after drainage rate of Q in mm/day and design rainfall P in mm for t in days is given by equation:

$$h = 40 + \frac{P}{3}t - Q \times t \quad (t \le 3 \text{ days}); \; h = 40 + P - Q \times t \quad (\text{for } t > 3 \text{ days})$$

Trail 1

Assume Q= 60 mm/day

Days	Initial depth (mm)	Design rainfall per day ($P_3/3$) mm (500/3 mm)	Discharge (Q) mm/day	Final water depth (mm)
1	40	166.67	60	146.67
2	146.67	166.67	60	253.33
3	253.33	166.67	60	360.00
4	360.00	-	60	300.00

Our assumption is maximum water level is 300mm which may persist for one day but here 300mm exceeds for 2 days hence we have to increase design discharge.

Trial 2

Assume Q = 70 mm/day

Days	Initial depth (mm)	Design rainfall per day(P3/3) mm	Discharge (Q) mm/day	Final water depth (mm)
1	40	166.67	70	136.67
2	136.67	166.67	70	233.33
3	233.33	166.67	70	330.00
4	330.00	-	70	260.00
5	260.00	-	70	190.00

Again maximum water level 300mm exceeds for 1 days hence we have to increase design discharge.

Trial 3

Assume Q= 80 mm/day

Days	Initial depth (mm)	Design rainfall per day(P3/3) mm	Discharge (Q) mm/day	Final water depth (mm)
1	40	166.67	80	126.67
2	126.67	166.67	80	213.33
3	213.33	166.67	80	300.00
4	300.00	-	80	220.00
5	220.00	-	80	140.00

Here maximum water level 300mm is persist for one day and 200 mm persist for up to 3 days hence our assumption is meet. Hence the design discharge for design of drainage is 80mm/day.

Here, Area = 40 ha

Discharge $Q = \dfrac{80 \times 10^{-3}\ \text{m}}{86400\ \text{sec}} \times 40 \times 10^4\ \text{m}^2 = 0.37\ \text{m}^3/\text{sec}.$

Bed slope = 1:400

Manning's roughness coefficient = 0.025

Assume, side slope = 1:1

B/D ratio = 3

Now, apply Manning's equation

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

$$A = BD + z\,D^2 = 3D^2 + D^2 = 4D^2$$

$$P = B + 2\,D\sqrt{2} = 5.828D.$$

$$R = A/P = 0.686D$$

Now put the values in Manning's equation and solve

D = 0.347m

B = 1.042m

Adopt free board = 0.1m

Or alternately we can assume non-silting and non-scouring velocity (depends on type of soil) instead of B/D ratio.

In this case assume velocity v = 0.8m/sec

Area $A = Q/v = 0.463m^2$

$A = BD + zD^2$ …… (i)

Calculate R from $v = \frac{1}{n} R^{2/3}\, S^{1/2}$

R = 0.253m

Now P=A/R = 0.463/0.253= 1.83m

We have, $P = B + 2\,D\sqrt{2}$ …….(ii)

Solving (i) and (ii)

B = 1.02m

D = 0.34m

Example 9.2

Determine the drainage rate in l/s/ha required to meet the following condition for healthy growth of rice paddies in bonded field in Terai of Nepal.

Initial water level in field = 50 mm

Maximum water level is 400mm which may persist up to one day.

Depth in excess of 250mm may persist for up to 2 days

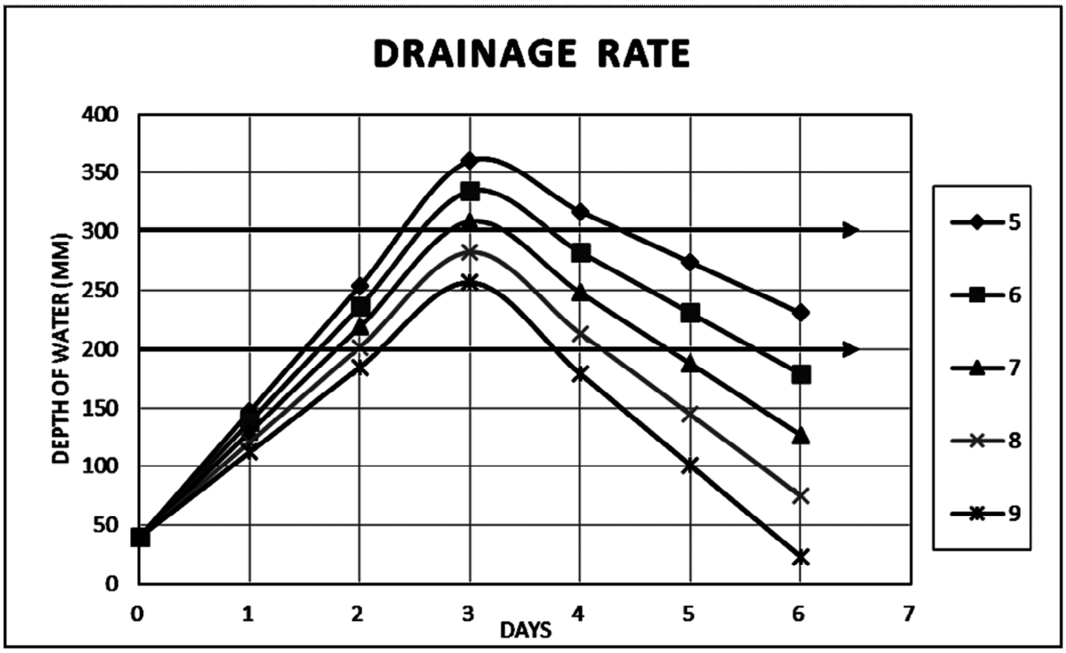
No rain follows the design rainfall for several days.

Neglect ET and deep percolation loss.

Design 3 day rainfall is 400 mm.

Solution:

Days	Q lps per ha	1	2	3	4	5	6	7
	Q mm per day	**8.64**	**17.28**	**25.92**	**34.56**	**43.2**	**51.84**	**60.48**
	Initial water depth (mm)	50	50	50	50	50	50	50
0	Rinfall P3/3							
1	133.333	174.693	166.053	157.413	148.773	140.133	131.493	122.853
2	133.333	299.387	282.107	264.827	247.547	230.267	212.987	195.707
3	133.333	424.08	398.16	372.24	346.32	320.4	294.48	268.56
4		415.44	380.88	346.32	311.76	277.2	242.64	208.08
5		406.8	363.6	320.4	277.2	234	190.8	147.6
6		398.16	346.32	294.48	242.64	190.8	138.96	87.12



Here from graph, our assumption i.e. maximum water depth 400mm persist up to one day and 250mm may persist for 2 days is met by 5 lps per hectare (43.2 mm per day).

Example 9.3:

How many days the field will be inundated above 200mm depth if a drainage rate of 3 l/s per ha is maintained by constructing internal drainage system? If three days design rainfall is 300mm. Will such system cause the depth to exceed 300mm?

Solution:

Assume initial depth of water= 40mm

Given Q		3	lps/ha	
		25.92	mm/day	
Days	Initial depth (mm)	Design rainfall per day(P3/3) mm (300/3)	Discharge (Q) mm/day	Final water depth (mm)
1	40	100.00	25.92	114.08
2	114.08	100.00	25.92	188.16
3	188.16	100.00	25.92	262.24
4	262.24		25.92	236.32
5	236.32		25.92	210.40
6	210.40		25.92	184.48

Here data shows that the field inundated above 200mm in 3 days and no any depth exceed 300mm.

Example 9.4:

Determine the drainage rate required to meet the following condition. Maximum yearly precipitation for three consecutive days = 300mm. The design rainfall is to be taken as 10 year return periods. Initial water level in field = 40mm. Maximum water level is 300mm which may persist for up to one day and depends in excess of 200 mm may persist for up to 3 days. Assume other suitable values if necessary.

Solution:

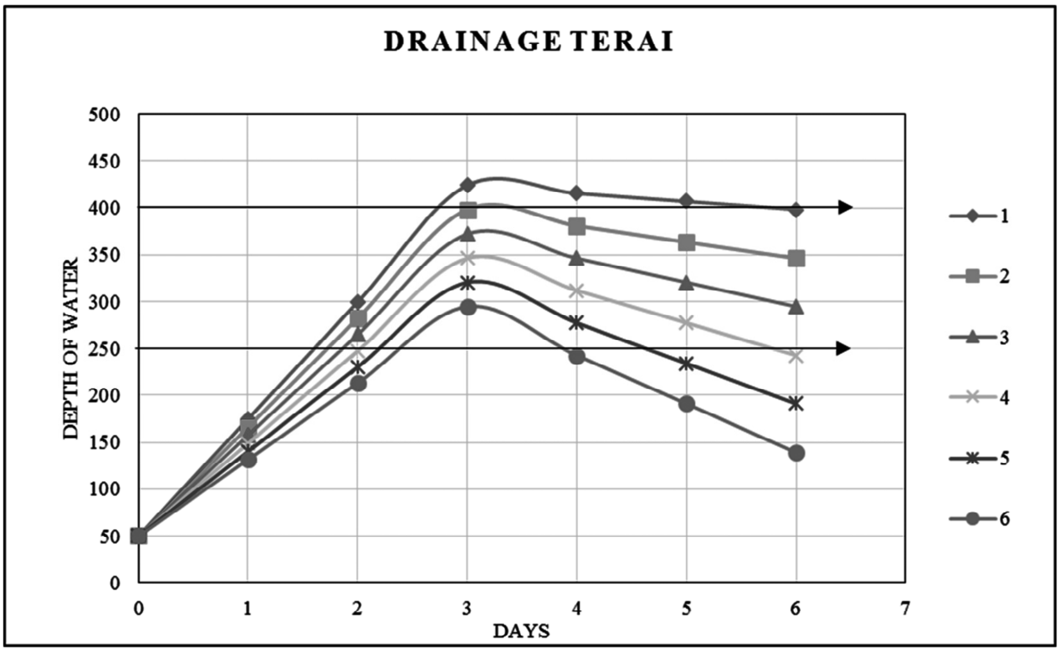
Here, maximum yearly precipitation for three days= 450mm

10 year design yearly precipitation for three days = growth factor x 300 mm

For 10 year return period, growth factor = G= 0.32 ln T + 0.78 = 1.5

Now design rainfall = 300x 1.5 = 450mm

days	Q lps per ha	5	6	7	8	9
	Q mm per day	43.2	51.84	60.48	69.12	77.76
	Initial water depth (mm)	40	40	40	40	40
0	Rainfall P3/3 (450/3) (mm)					
1	150	146.8	138.16	129.52	120.88	112.24
2	150	253.6	236.32	219.04	201.76	184.48
3	150	360.4	334.48	308.56	282.64	256.72
4		317.2	282.64	248.08	213.52	178.96
5		274	230.8	187.6	144.4	101.2
6		230.8	178.96	127.12	75.28	23.44



From graph and table our assumption i.e maximum water depth 300mm for one day and 200 mm may persist for 3 days is meet by 8 lps per ha. Hence designed drainage rate is 8 lps per ha.

1. **Hills**

   The internal drainage of rice fields in the hills will vary often take the form of spillage from terrace to terrace via natural channel through the bunds. In some cases, however, field drains will be necessary, and these may be designed on the basis of runoff estimated for a water balance similar to that described for the Terai. The following assumptions are made:

   - Initial water level is 40 mm
   - Maximum water level is 100 mm.
   - No rain follows the design rainfall for several days.
   - Losses due to evapotranspiration and deep percolation are replaced by ongoing irrigation and/or flood inflows during the design rainfall.

   The balance may be expressed:

   $$Q = P + 40 - 100 = P - 60$$

   Where, Q is the runoff in mm

   P is the design rainfall in mm.

   The 24 hrs rainfall with a return period of 10 years is recommended for the design of field drains in the hills.

9.5.2 External Drainage

External drainage refers to runoff generated outside the irrigation area which presents a danger to the irrigation scheme. It covers the assessment of design floods for the headworks and the assessment of stream flows, which must be safely passed through or around the irrigated area, i.e., cross drainages. Peak external drainage rate are required to design structures to carry runoff safely across or around the irrigated area.

9.5.3 Remodeling of existing natural drains

In areas where it is intended to use existing natural drains, it will be necessary to check that their capacity is sufficient to carry the drain design discharge. The first task would be to survey the channel, taking cross – section at a maximum spacing of 250m. These would be plotted and a longitudinal section of the drain produced at a scale of 1:5000, showing typical bed levels and ground levels adjacent to the drain. It is important that the bed levels plotted on the longitudinal section are representative of the average bed level in each section (M.9_Drainage, 1990).

A straight line or series of straight lines would then be fitted to the bed levels on the longitudinal section. The cross section should be transformed into trapezoidal channels of similar form. The drain capacity below the design water level should than be calculated using the manning's equation.

However, the existing natural drain may be very irregular in both level and in its cross – sectional shape and size, and any process of approximation to a uniform channel may be unrealistic. In this case it is necessary to carry out a back water analysis of the channel. The standard step is normally used for channels with a varying cross – section.

It is normal to start at a scion where the water level for the design discharge is known. This is normally a structure at or near the downstream end of the drain. The calculation is performed by trial and error for successive steps (distances between cross – section) to give the water level at each cross – section, working from downstream to upstream. These water levels can then be compared with the required design water levels in the channel and hence used to determine if the channel has sufficient capacity to carry the design discharge.

If it is necessary to increase the capacity of the natural drain this can be done in a number of ways:

- Increase the depth of the drain.
- Increase the width on one side or both sides.
- Increase both the width and depth.

The choice of solution is dependent on the constraints of the existing channel. Any structures across the channel may make it difficult to deepen the channel. Buildings or structures located close to the banks of the channel may limit the increase in channel width. Should the channel be constrained by buildings say, through a village, it may be possible to culvert or flume the channel through that village. Each individual drains should be examined for the best solution and solution, and several method may be used on one drain in different reaches. Once the slope and levels of the new channel are decided, the channel is designed in the normal manner. It may not be possible to retain the standard B/D ratio and flexibility in this respect is required.

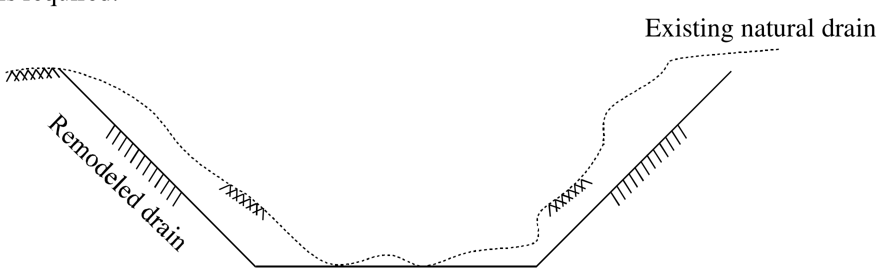


Figure 9.4: Remodeling of existing natural drains

9.6 Sub – Surface drainage system and their design

Sub – surface drain or tile drains are required for high water table conditions in the areas where the soils are having poor internal drainage ability. If no impervious layer occurs below the farm land and the water table is low (lower than about 3m from the ground), internal soil drainage may be sufficient and no tile drains needed. For maximum productivity of most of the crops, both surface as well as sub-surface drains may sometimes, however, become, essential, particularly in areas of higher water table.

Advantages of Tile Drain

Tile drainage helps in increasing crop yields by draining the water or by lowering the water table in the following manner:

a. Removes the free gravity water, not available directly to the plants, thus, increase the volume of root zone soil from which roots can obtain nutrient.

b. Increase air circulation and bacterial activity in the soil, thus improving soil structure and making the plant food more readily available.

c. Reduces soil erosion as a well-drained soil has more capacity to hold rainfall, resulting in reduced runoff and soil erosion.

d. Helps removing toxic substances such as sodium, their excess amount may retard plant growth.

e. Lowering the water table during rainy seasons thus checking water logging and salinity and other soil and water problems.

f. Easy and timely cultural practices.

Limitations:

a. Providing underground tile drains is a costly affair and may be required only in areas of high water table, and where the ground soil has a poor internal drainage capacity, in other words where it is absolutely desired.

b. Without proper maintenance and care it becomes nonfunctional.

9.6.1 Layout of subsurface drainage system

Tile drains are usually, pipe drains made up of porous earthenware and are circular in section. The diameters may vary from 10 to 30 cm or so. These drains are laid below the ground level, butting each other with open joints. The trenches in which they are laid are back filled with sand and excavated material to form an envelope.

The drain should be aligned to follow the paths of the natural drainage. Every drain should be provided with an outfall, either into a bigger drain or natural stream. The location of the outfall is adjusted to obtain the required slope in the drain.

A closed drain system in general consist of a number of lateral drains (also lateral or branch drains) which collect water from different areas and discharge into a main drain. The main drain is usually an open drain. Sometimes lateral discharge into a sub main which is usually a closed drain and the sub main discharges into main drain. The main drain in turn discharge into outfall drain which takes water to natural drain. The layout of a closed drain system is governed by the topography of the area to be drained. The various layout plans for closed drain system are shown in Figure. 9.5 And are discussed below.

1. Natural system

In this system the main/sub main and the connecting laterals are provided in natural courses as shown in figure 9.5 (a). The natural system is preferred in rolling topography.

2. Grid iron layout

In this system the laterals are provided only on one side of the main / sub main as shown in figure 9.5 (b). This system is used when the land is practically level or where the land slopes away from the main/ sub main on one side and when the entire area has to be drained.

3. Herringbone pattern

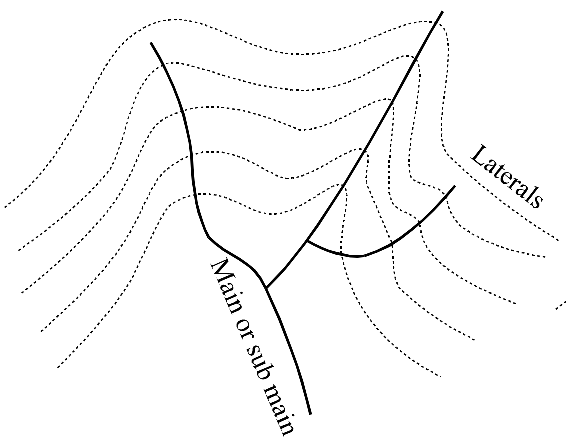
In this system laterals join main / sub main from each side alternately as shown in Figure 9.5 (c). It is adopted when the main /sub main is laid in a depression. In this case the land along the main / sub main is double drained which is however, necessary because it is in depression and hence requires more drainage than the land in the adjacent slopes.

4. Double main system

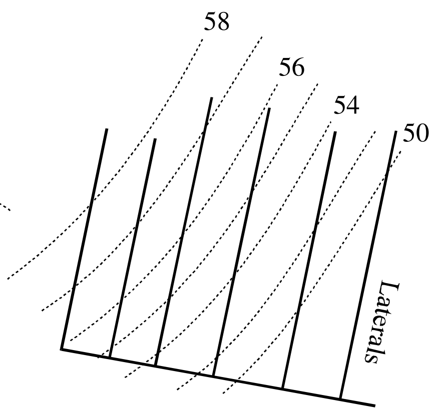
In this system two separate mains/sub mains with separate laterals for each are provided as shown in Figure 9.5(d). This system is provided when the bottom of the depression is wide. This arrangement helps to reduce the length of the laterals and eliminates the break in slope of the lateral at the edge of the depression.

5. Intercepting drain system

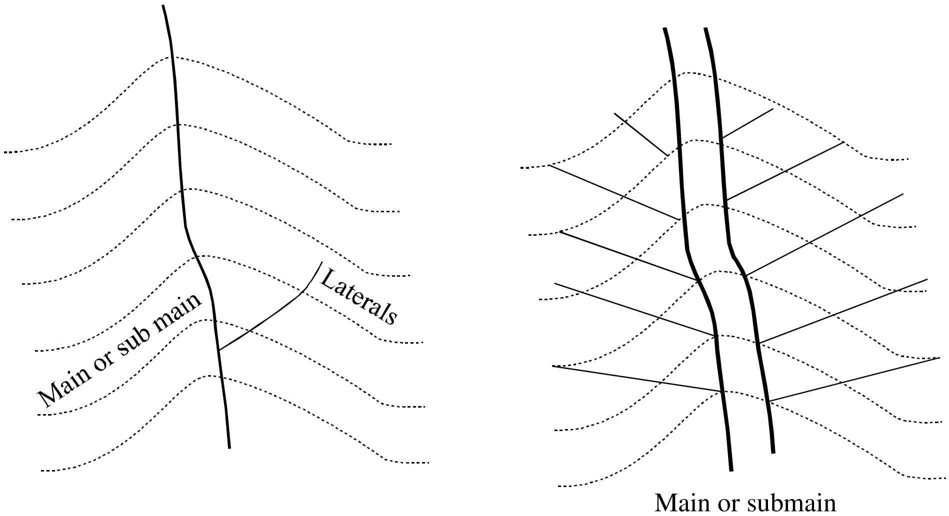
In this system there are no laterals but only a main/ sub main is provided at the toe or the slope as shown in Figure 9.5 (e). This arrangement is adopted when the hilly land is to be drained.



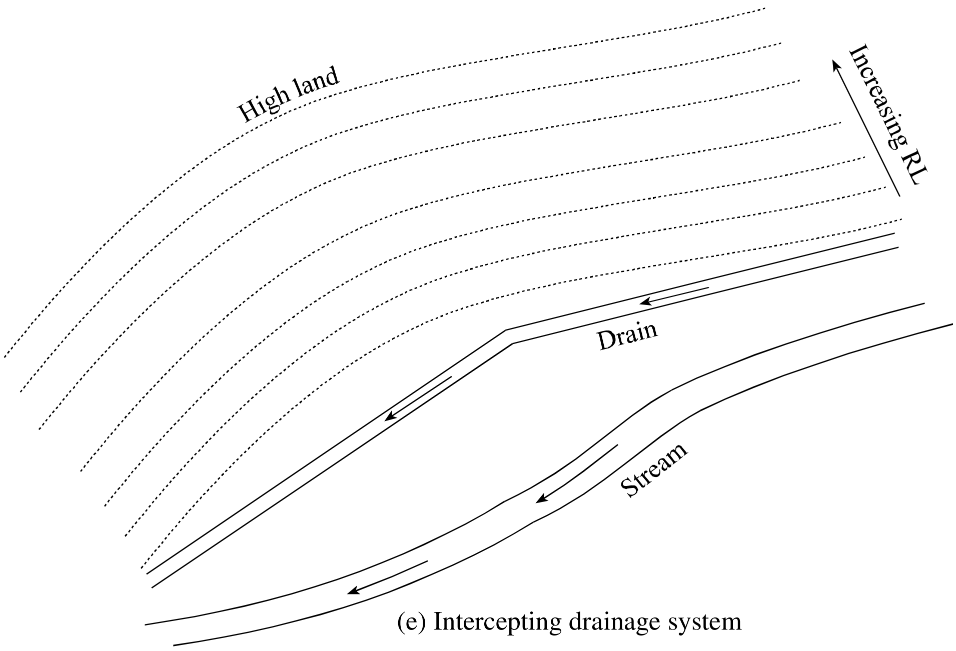
(a) Natural system



(b) Grid-Iron system



(c) Herringbone pattern (d) Double main system



(e) Intercepting drainage system

Figure 9.5: Layout plan of Tile drains

9.6.3 Flow of ground water to drain and spacing of tile drains

After proper installation the tile drains starts removing water movement of water can be summarized as follows:

1. If soil is fully saturated, water flows into the tile drain along the path shown in Figure 9.6**.** Water as farther distance has to travel more to get drained.

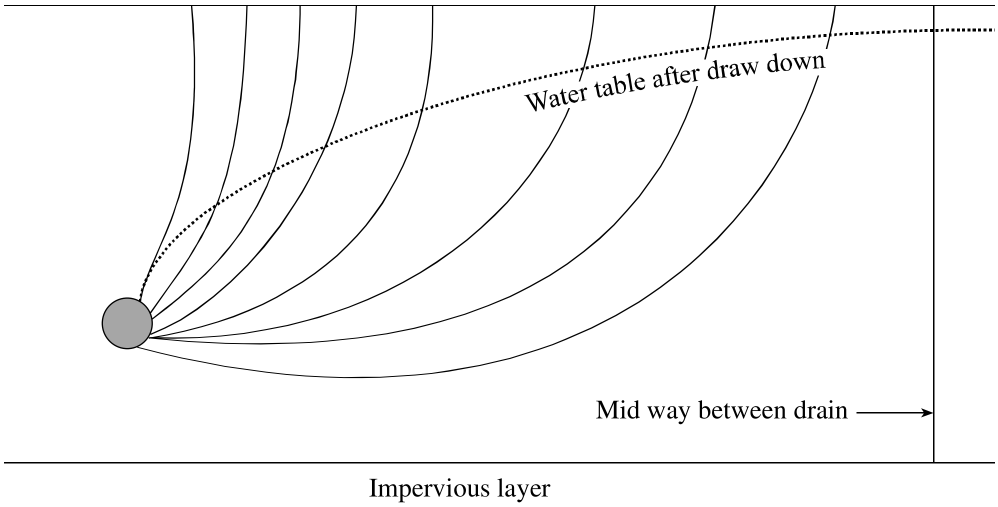


Figure 9.6: movement of water to drain

2. The falling of water table, i.e. draw down will be more near the tile than at the points farther away Since the quantity of water moving between any two flow lines is the same.
3. Once the saturated soil has drained for a day or two, the resulting water table will be, as shown in Figure 9.6. With the passage of time, the water level goes on lowering down.
4. With series of tile drains, the sub soil water level directly over the drains, is lower than the level midway between them as in shown in Figure 9.6.
5. When a filter is provided around the tile drains to surround the drains with more pervious soil, then the overall draw down will be more.

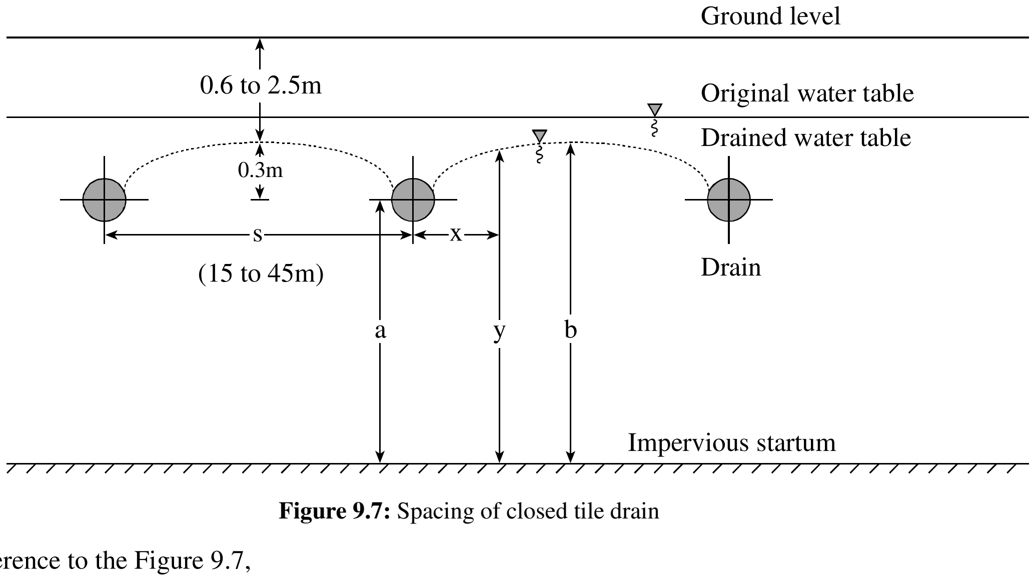


Figure 9.7: Spacing of closed tile drain

With reference to the Figure 9.7,

Let s = Spacing of the drains.

a = height of drain above the impervious stratum

b = maximum height of the drained water table above impervious stratum

y = height of impervious stratum at x distance from the center of drain.

Assumptions

- The hydraulic gradient at a distance x from the center of the drain is $\frac{dy}{dx}$.
- Flow line are parallel and area of flow section at distance x from the center of the drain per unit length of the drain is $y \times 1 = y$.
- Discharge towards the drain is inversely proportional to the distance from the drain.

From Darcy's law

$$Q = K.i.A \qquad \text{. . . 9.1}$$

Where $i = \frac{dy}{dx}$; $A = y$

For per unit discharge $q = k \times \frac{dy}{dx} \times y$ . . . (i)

When $x = \frac{s}{2}$; $q_y = 0$

When $x = 0$; $q_y = \frac{q_d}{2}$; q_d = Total discharge per unit length.

Thus; $q_y = \frac{q_d}{2} \frac{s/2 - x}{s/2}$

$$q_y = \frac{q_d}{2s} (s - 2x) \qquad \text{. . . (ii)}$$

Equating (i) and (ii)

$$q_y = \frac{q_d}{2s} (s - 2x) = k. \frac{dy}{dx} .y$$

$$\frac{q_d}{2Ks} (s - 2x) = y \frac{dy}{dx}$$

Integrating we get

$$\int \frac{q}{2Ks} (s - 2x) \, dx = \int y \, dy$$

$$\frac{q_d}{2Ks} (sx - x^2) = \frac{y^2}{2} + C \qquad \text{. . . 9.2}$$

When $x = 0$; $y = a$. then $C = -\frac{a^2}{2}$

Making this substitution and simplifying we get;

$$q_d = \frac{sK\,(y^2 - a^2)}{sx - x^2} \qquad \text{. . . 9.3}$$

Also when $x = \frac{s}{2}$; y = b, from equation 9.3

$$q_d = \frac{4K\,(b^2 - a^2)}{s}$$

$$s = \frac{4K\,(b^2 - a^2)}{q_d}$$

Example 9.5:

In a drainage system closed drains are to be placed with their center 2 m below the ground level to keep the highest position of the water table 1.7m below the ground level. The impervious stratum is at a depth of 9.6 m below the ground level. If 1% of the average annual rainfall to be drained in 24 hours and co – efficient of permeability K = 1x 10^{-5} m/sec. If the average annual rainfall in the area is 850 mm. find the spacing of drain.

Solution:

Spacing $s = \frac{4K\,(b^2 - a^2)}{q_d}$

$K = 1 \times 10^{-5}$ m/sec

b = 9.6 – 1.7 = 7.9 m

a = 9.6 – 2 = 7.6 m

1% of average annual rainfall over and area A m^2 supply by each drain $= \frac{850}{1000} \times A \times \frac{1}{100}$

$= 85 \times 10^{-4} A\ m^3$

This is to be drained in 24 hrs and hence,

Volume of water to be drained off per second $= \frac{85 \times 10^{-4} A}{86400} = 9.8379 \times 10^{-8}\ A\ m^3/sec$

If s is the spacing between the drain then area A supply by each drain per unit length of the drain is; A= s × 1 = s

$q_d = 9.8379 \times 10^{-8}\,s$

Substituting these values $s = \frac{4 \times 1 \times 10^{-5}\,(7.9^2 - 7.6^2)}{9.8379 \times 10^{-8}\,s}$

On solving, s = 43.487 m

Hence drains should be provided at a spacing of 43.5 m.

Exercise 9:

1. What is water logging? Explain causes and preventive measures of water logging.
2. What are the effects of water logging?
3. What is internal drainage and External drainage?
4. Explain about the importance of surface and subsurface drainage in Terai as well as Hills of Nepal.
5. What is surface and subsurface drainage?
6. Explain the importance of remodeling of existing drainage.
7. Write the steps to design surface drainage in the bunded field of Terai.
8. What are the assumptions to design surface drainage in Terai and Hills of Nepal?
9. Design a surface drainage for a field of 50 ha area in Terai with following data. Design maximum yearly precipitation for three consecutive days = 450mm, longitudinal slope of channel 1:400, manning roughness coefficient 0.02, permissible velocity is 0.7m/sec. Maximum water level is 300mm which may persist for up to one day and depends in excess of 200 mm may persist for up to 3 days. Assume other suitable values if necessary.
10. How do you determine the spacing of tile drain, explain with derivation.
11. Find the spacing between drains for the following data
    - Annual rainfall = 1000 mm
    - Height of drains above impervious stratum = 4.5m
    - Maximum height of the drained water table above the impervious stratum= 5.0m
    - Coefficient of permeability k = 10^{-6} m/sec.
12. Find the inflow into the drain for the following data:
    - Spacing = 13m
    - Height of drain above impervious stratum= 4.5 m
    - Maximum height of the drained water table above the impervious stratum = 5.0m
    - Coefficient of permeability k = 10^{-6} m/sec
13. The annual rainfall in Biratnagar is 2000 mm. Find the spacing of subsurface drain if 2% of average annual rainfall is to be drained in 2 days.

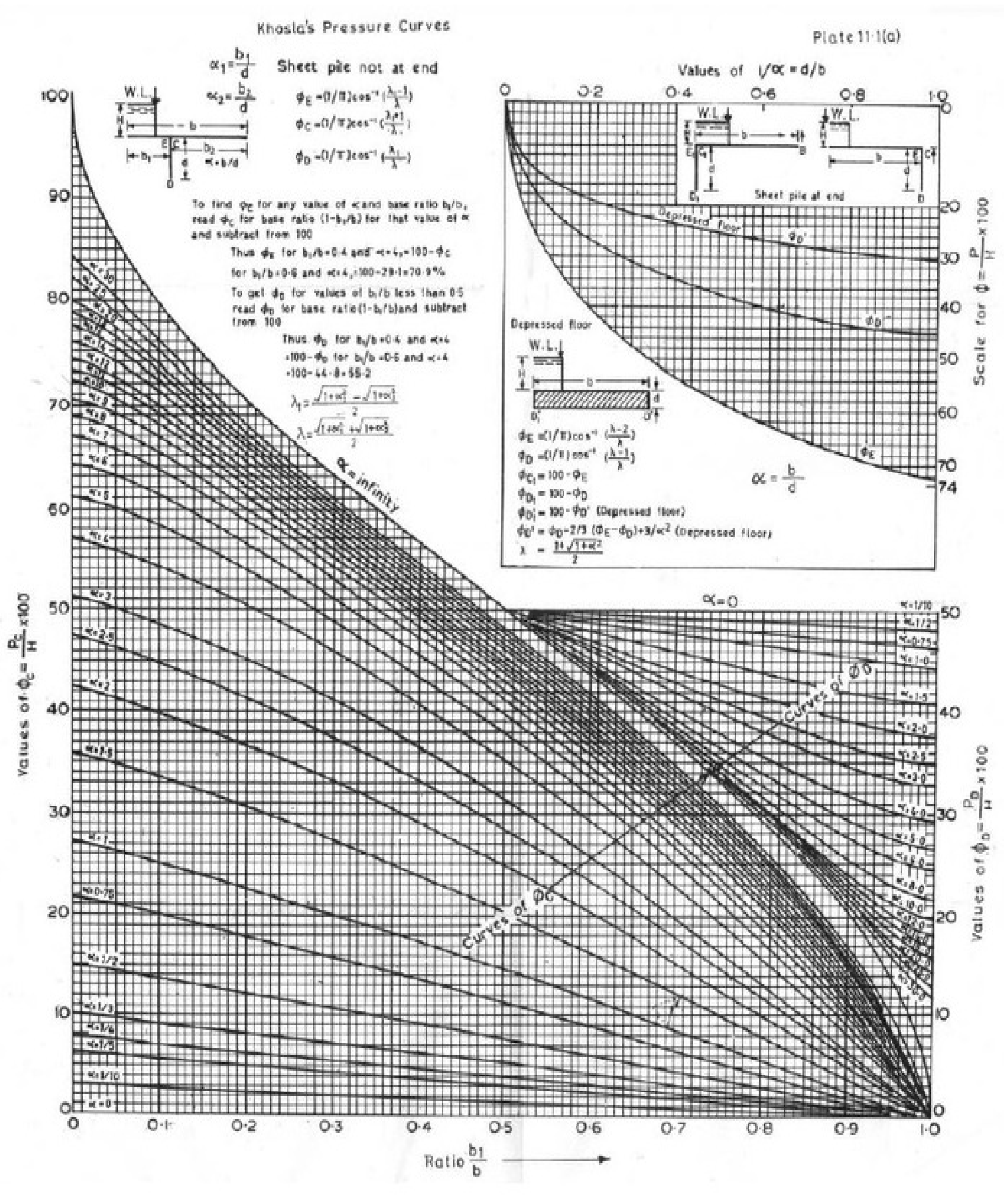
    Given;

    Depth of impervious stratum from the top of soil surface = 12m

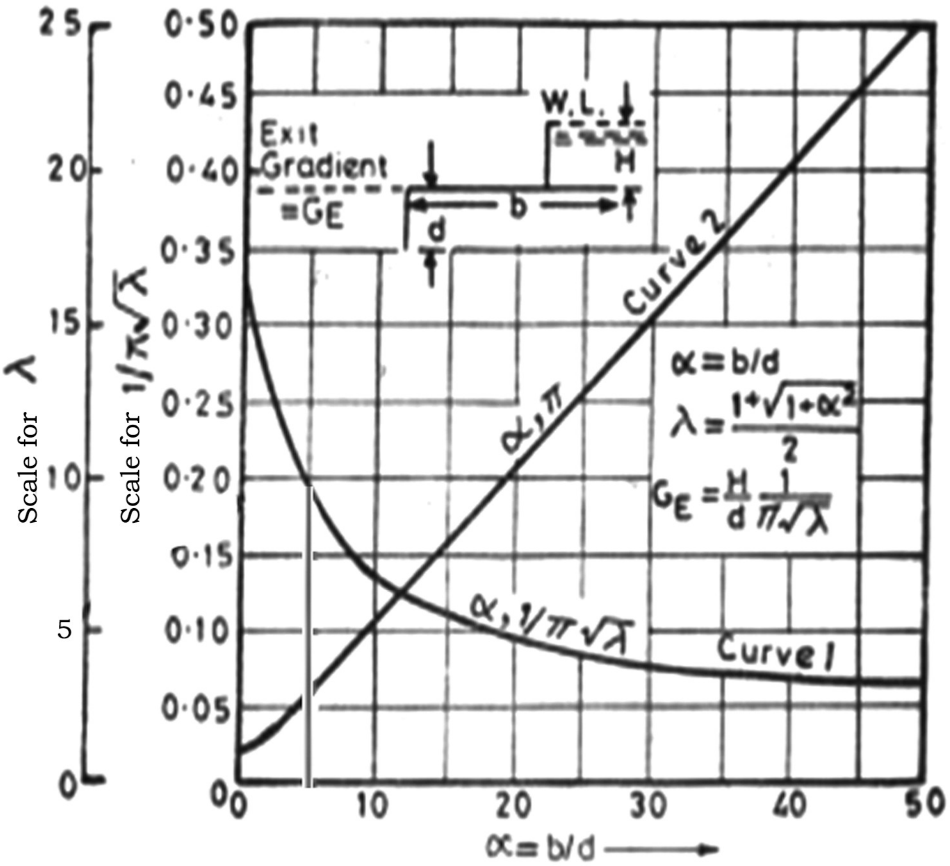
    Position of drain is 2m below the top soil surface and the depth of highest position of water table below the top soil surface = 1.5m.

    Permeability, K = 1×10^{-4} m/sec.

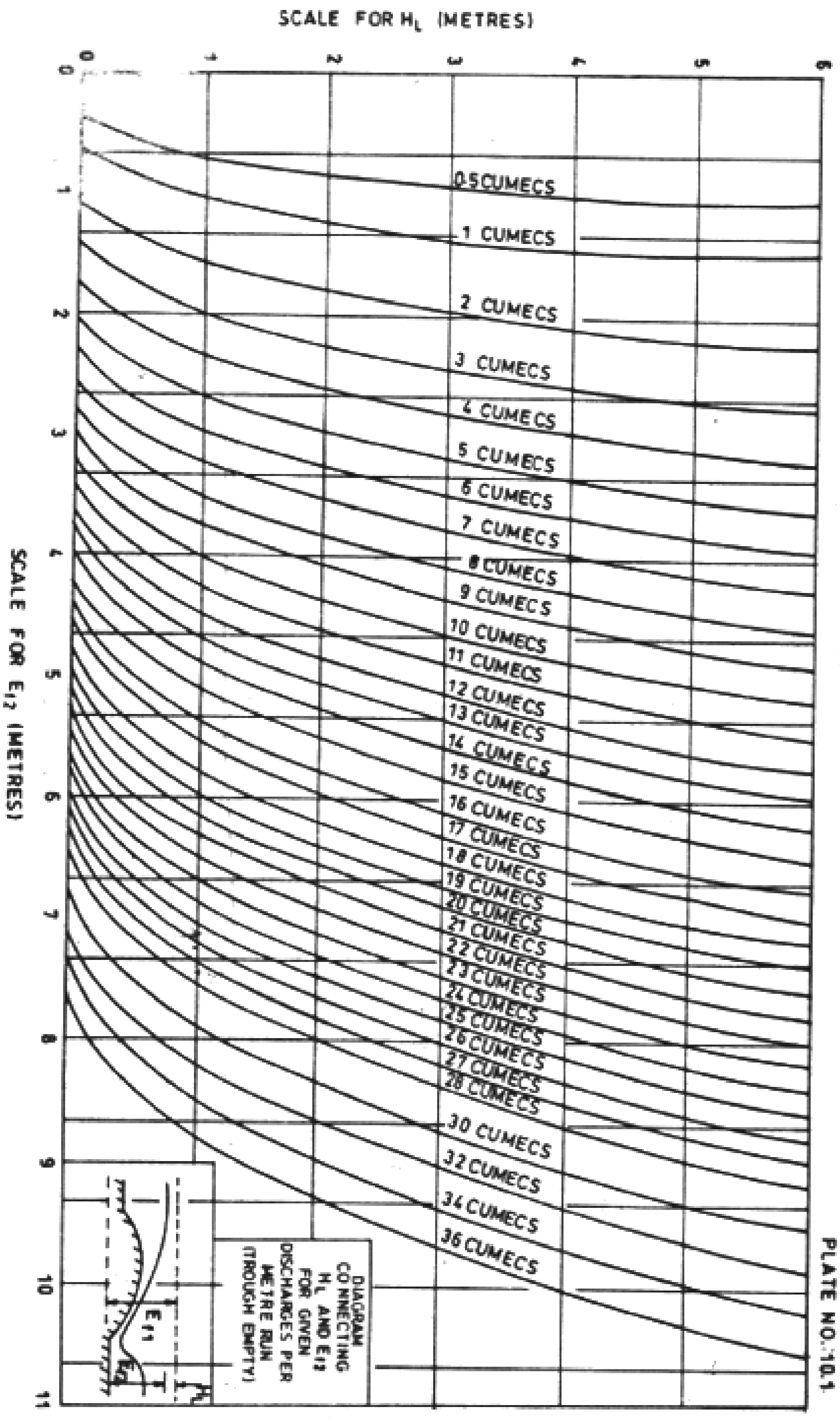
Annex I


Khosla's Pressure Curves
Plate 11·1(a)
Sheet pile not at end
Values of 1/α = d/b
Sheet pile at end
Depressed floor
α = infinity
α = 0
Curves of φC
Curves of φD
Values of φC = PC/H × 100
Values of φD = PD/H × 100
Scale for φ = P/H × 100
Ratio b1/b
To find φE for any value of α and base ratio b1/b, read φC for base ratio (1-b1/b) for that value of α and subtract from 100
To get φD for values of b1/b less than 0·5 read φD for base ratio (1-b1/b) and subtract from 100

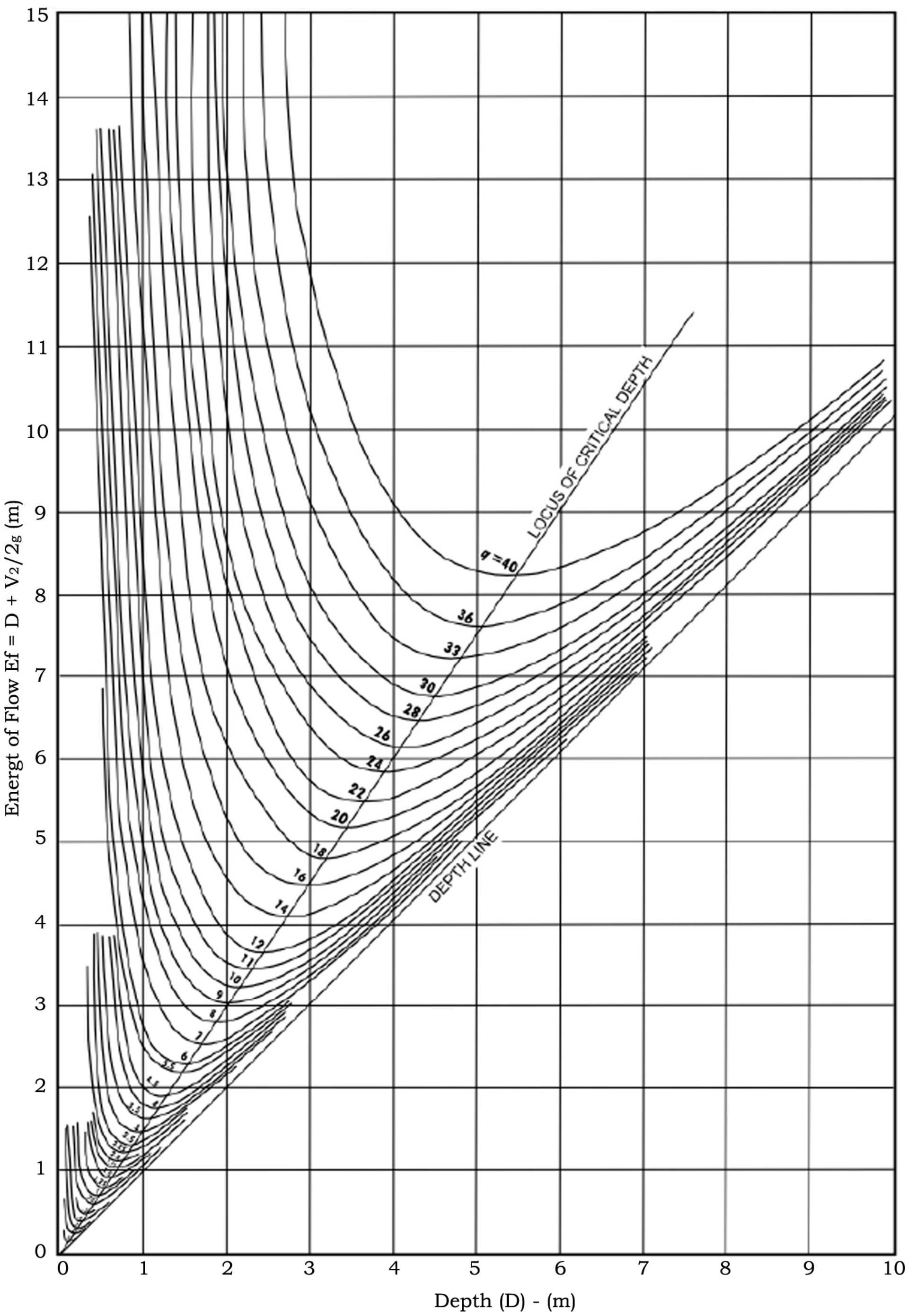
Annex II


Scale for λ
25
20
15
10
5
0
Scale for 1/π√λ
0·50
0·45
0·40
0·35
0·30
0·25
0·20
0·15
0·10
0·05
0
0
10
20
30
40
50
α = b/d
Exit Gradient = GE
W.L.
H
b
d
Curve 2
α, π
Curve 1
α, 1/π√λ
α = b/d
λ = (1+√(1+α²))/2
GE = H/d · 1/(π√λ)

Annex III

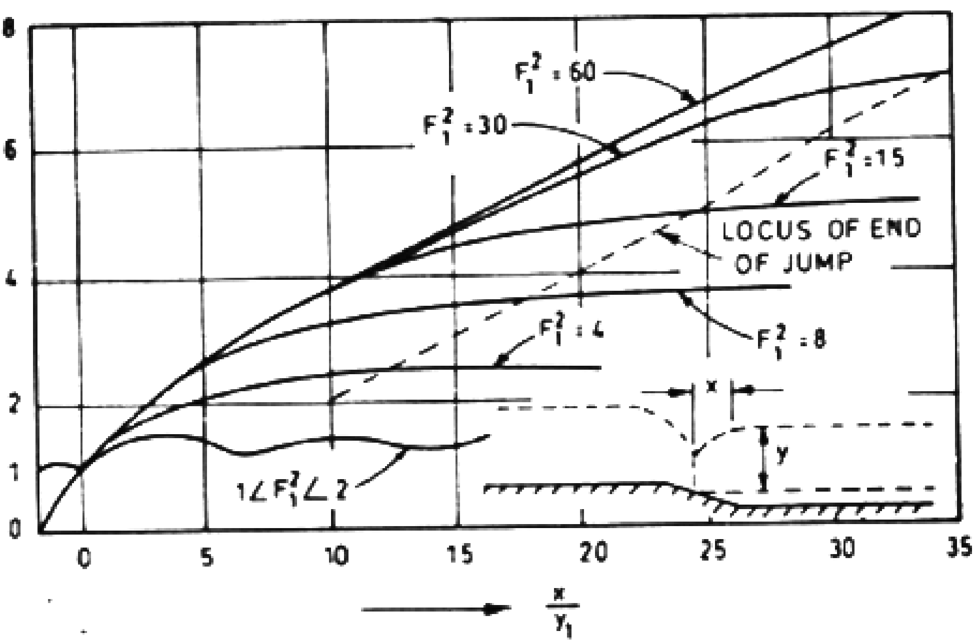

SCALE FOR H_L (METRES)
SCALE FOR E_f2 (METRES)
0.5 CUMECS
1 CUMECS
2 CUMECS
3 CUMECS
4 CUMECS
5 CUMECS
6 CUMECS
7 CUMECS
8 CUMECS
9 CUMECS
10 CUMECS
11 CUMECS
12 CUMECS
13 CUMECS
14 CUMECS
15 CUMECS
16 CUMECS
17 CUMECS
18 CUMECS
19 CUMECS
20 CUMECS
21 CUMECS
22 CUMECS
23 CUMECS
24 CUMECS
25 CUMECS
26 CUMECS
27 CUMECS
28 CUMECS
30 CUMECS
32 CUMECS
34 CUMECS
36 CUMECS
DIAGRAM CONNECTING H_L AND E_f2 FOR GIVEN DISCHARGES PER METRE RUN (TROUGH EMPTY)
PLATE NO. 10.1

Annex IV


Energt of Flow Ef = D + V2/2g (m)
Depth (D) - (m)
LOCUS OF CRITICAL DEPTH
DEPTH LINE
q =40
36
33
30
28
26
24
22
20
18
16
14
12
11
10
9
8
7
6
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15

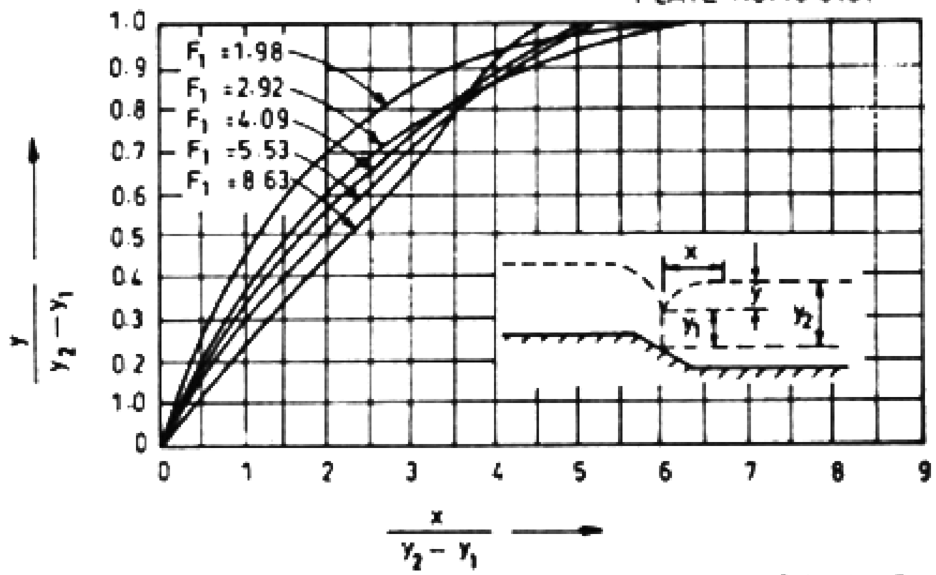
Annex V

PLATE NO 10.3 (a)



CURVE FOR PLOTTING POST JUMP PROFILE

PLATE NO.10.3(b)



ALTERNATE CURVE FOR PLOTTING POST JUMP PROFILE