Content

1. Syllabus ………………………………………………………….. 1/3
2. Question Bank ………………………………………………3/11
3. Introduction……………………………………………………11/107
2 Concept and Principles……………………………………..107/177
3 Planning and Design…………………………………………177/700
4 Management and Other Related Aspects……………………859/875
5 Engineering Costing and Economic Analysis……………… 875/903
6 Construction Management………………………………………903/925
7 Maintenance and Rehabilitation……………………………..925/932
8 Environmental Impact Assessment……………………………..932/948
9 others if any ……………………………………………………948/971

लोक सेवा आयोग

नेपाल इन्जिनियरिङ्ग सेवा, सिभिल समूह, इरिगेशन उपसमूह, राजपत्रांकित प्रथम श्रेणी (सहसचिव वा सो सरह पद) को खुला र आन्तरिक प्रतियोगितात्मक लिखित परीक्षाको पाठ्यक्रम

द्वितीय पत्र सेवा सम्बन्धी प्राविधिक विषय

पूर्णाङ्क :- १००

Introduction

Water sector policies and strategies with respect to irrigation development

Major development projects and economic policies affecting the irrigationsector

History of irrigation development in Nepal

Impact of irrigation development in the poverty reduction strategy

Strategy and policies for the mitigation and management of floods

Evolution of policies shifting from agency managed to farmer managed system

Overview of Irrigation development in the SAARC region

Irrigation development in current plan period

National economy and irrigation development strategy

Resource mobilization in irrigation sector

External financing in irrigation sector

Concept and Principles

Issues and Problems of irrigation development in Terai and hills

Design, philosophy and approaches to different types of irrigation systemdevelopment

Design, philosophy and approaches of disaster mitigation with reference tolandslide and floods

Concept of bio-engineering for watershed management
Integrated Water Resources Management and Role of Irrigation

Planning and Design

Surface Irrigation System

Designing of Head Works
Design Consideration of canals, canal structures and cross drainage
Hydrology and Agro-meteorology
Design philosophy and approaches

Water availability for diversion crop water requirement; and waterrequirement at headwork.

Sediment transport and its effects on head works
Farm drainage planning and design

Ground Water System

Ground water development planning
Ground water hydrology and construction of tube wells.
Sediment Transport and its effects on headworks
Farm Drainage planning and Design
Design consideration for deep and shallow tube wells

Multipurpose Projects

General planning of multipurpose reservoirs
Capacity of reservoir
Selection of suitable site for a reservoir
Flood routing or flood absorption.
General design considerations for dams
Criteria for structural stability of gravity dams.
Design consideration of spillways and its types
Aeration arrangement in gated and overflow spillways

River Training Works

River morphology, design, concept of different types of river trainingstructures
Concept of sediment transport and its influence on river trainingworks
River training and bank protection principles
Inundation Problem in Nepal

Management and Other Related Aspects

Water resources management planning, opportunities, threats, organization,actuation, and controlling
Watershed management and integrated water use plan
Participatory irrigation management in Nepal
Ground water management
Private sector involvement and contracting

Engineering Costing and Economic Analysis

Different approaches to cost estimates
Economic analysis of project
Economic concept

Construction Management

Classification of irrigation projects
Methods and construction technology
Contract management and quality aspect of construction
Monitoring and evaluation at each level of project
Quality control and assurance

Maintenance and Rehabilitation

Maintenance management system and its analytical approach
Maintenance concept and methods
Personnel, plant and equipment management
Process and performance of irrigation management
Participatory management, collection of irrigation service fee
Transfer of operation and management to private party
Contract disputes resolution
Concept of value engineering
Subsidy issues

Environmental Impact Assessment

Concept of environmental assessment, Initial Environmental Examination (IEE), Environmental Impact Assessment (EIA) and role of EIA, types of environmental impacts and principles

Environmental and social impact assessment in irrigation projects and its mitigation

Environmental impacts assessment methodology: screening, scooping, and initial impact identification, Initial Environmental Examination (IEE), TOR preparation and writing EIA report

Management of EIA procedures: public participation, EIA review, mitigation measures and monitoring

Environmental auditing.

- - - -

Old Question

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२०७८/10/06 गते

समय –: ३ घण्टा पूर्णाक –: १००

पत्र –: दित्तिय बिषय –: सेवा सम्बन्धि प्राविधिक बिषय

तलका प्रश्नको उतर छुटाछुटै उतरपुस्तिकामा लेख्नुपर्नेछ अन्यथा उतर पुस्तिका रद्द हुनेछ ।

1. How do you evaluate the present status of irrigation development in Nepal? What factors in your opinion shall be considered to increase the efficiency of the present irrigation systems? 15
2. Describe briefly about Integrated Water Resources Management (IWRM) and explain the linkage of land and water governance, poverty and sustainability. 15
3. What are the basic steps of Environmental Impact Assessment and Initial Environmental Examination? List the Procedure followed in each of these steps. 15
4. What are the problems in mustering equity for irrigation project in Nepal? Suggest a model for generating public investment in irrigation projects ? 15
5. Discuss the pros and cons of Peoples Embankment Program in Nepal. How the gaps in implementation of such programs can be brides for the sustainable solution to inundation and bank cutting issues in Nepal. 20
6. Department of Water resources and irrigation in developing inter basin water transfer multipurpose projects in different parts of Nepal. How do you resolve the conflicts among provinces in benefits sharing and water right? Describe the different phases in this type of projects. 20

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२०७६/९/१२ गते

समय –: ३ घण्टा पूर्णाक –: १००

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1. It has been found that monitoring and evaluation of irrigation projects are not carried out properly. Why monitoring and evaluation is not given due priority? Mention about the importance of monitoring and evaluation at each level of projects for successful implementation. 15
2. What are the major problems for irrigation development in Terai and Hills of Nepal? Suggest measures to overcome those issues. 15
3. Describe briefly integrated water resources management and the role of irrigation in it. What are the design considerations for deep and shallow tube wells? 10+5
4. What is the status of surface and ground water irrigation development in Nepal? How Nepal can take benefits from conjunctive use of surface and ground water? 15

5. Give a glimpse of the history of development of irrigation in Nepal. How successful/unsuccessful have been the national investments in irrigation project? Discuss the reasons behind and suggest measures to be taken at policy level for the improvement in present state of irrigation development in Nepal.

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२०७३/८/१२ गते

समय –: ३ घण्टा पूर्णाक –: १००

पत्र –: दित्तिय बिषय –: सेवा सम्बन्धि प्राविधिक बिषय

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1. asing externalThe decreassistance and increasing cost of construction to meet the financial requirement , in which way the financing for irrigation project can be done by mobilizing own resources be mobilized from the developed irrigation project? 15
2. Discuss the problems in collection of irrigation service fee. Discuss what do you understand by environmental auditing of irrigation projects? 15
3. Sediment transport has remained a major problem in planning, design and management of irrigation project in Nepal. It is crucial in determining the life of the head works. Discuss various factors that affect sedimentation. 15
4. Explain the scope of value engineering in the development of irrigation. Can the design policies be changed by applying the value engineering techniques in Nepal? 15
5. what are the main problems in execution of irrigation projects under external financing like WB ADB? Suggest the measures to overcome those problems. 20
6. water is a precious natural resource in Nepal. However, its utilization has been far below the requirement. NWRS and the National Water Plan has developed strategic and indicative plan for future water development in the country. Elaborate the details of the plan. Analyze its do-ability and suggest measures if any for its successful implementation. 20

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२०७२/८/२० गते

समय –: ३ घण्टा पूर्णाक –: १००

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1. Discuss about the inundation problems in Nepal and also give their respective solutions. Point out the advantages and disadvantages of participatory irrigation management in Nepal. What type of economic analysis is required for an irrigation project? 5+5+5
2. Explain about the construction technology of irrigation projects. How irrigation service fee (ISF) can be properly collected from the irrigation projects in Nepal? What is Environmental and social Impact Assessment (ESIA) and its mitigation in irrigation projects? 5+5+5

3. Describe in detail ground water potential and its development plan in Nepal. What could be the advantages and disadvantages of surface and ground water for irrigation in hills and terai of Nepal? 15
4. Elucidate the need of hydrological and agro-meteorological studies in the design of surface and ground water irrigation system. 15
5. State with examples and case studies about the possibility of private sector's involvement in irrigation management in Nepal. 20
6. Irrigation management transfer (IMT) has been taken as an important management tool in irrigation policy of Nepal. However, experience shows that the management transferred irrigation projects to WUAs are still not taking its pace as envisaged by the irrigation policy. What are the key elements needed for the success of IMT?

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२०७१/८/१८ गते

समय –: ३ घण्टा पूर्णाक –: १००

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1. Describe the maintenance management system currently adopted in the irrigation systems in Nepal. What is your opinion about the annual budget allocated for operation and maintenance of irrigation systems in Nepal? 15
2. Describe how land slide and flood related disasters occurs. Explain how they can be controlled /mitigated in the context of Nepal. 15
3. What are the possibilities and limitations of ground water use for irrigational purpose in different parts of Nepal? suggest precautionary measures for optimization of ground water use. 5+5+5
4. Describe briefly the main features of water resources strategy, 2002. List the strategic output for the use of the water resources of Nepal. 15
5. कमला सिंचाई आयोजना र बागमती सिंचाई आयोजना एकै प्रकारको भौगोलिक अवस्थितिमा रहेका छन् |दुवै नदीको Bedmaterial पनि समान आकारप्रकारका छन् |यी दुवै आयोजनाको headwork निर्माण भएको केही दशक भित्रै weir को crest खिइएर उचाई)weirheight (घट्दै गएको छ |परिणाम स्वरूप designedpondlevelmaintain गर्न सकिएको छैन र सिंचित क्षेत्र् घट्न गएको छ |समस्यालाई विश्लेषण गरि यसो हुनका कारणहरु के के हुन सक्दछन् ?समाधानका लागि कस्ता उपाय अवलम्बन गर्नुपर्ला ?सुझाव प्रस्तुतु गर्नुहोस् |२०
6. गण्डक सिंचाई आयोजना व्यवस्थापन हस्तान्तरण नीति अन्तर्गत उपभोक्ता समितिलाई दशक आगाडी हस्तान्तरण गरिएको थियो |हाल आएर यस आयोजना लाई पुनःसिंचाई विभागलाई नै हस्तान्तरण भैसकेको आयोजना पुनःनेपाल सरकारले लिनुपर्ने अवस्था आएको छ |प्रस्तुत समस्याको गाम्भीर्यता लाई मनन गरि यसो हुनका करान्हारू के के हुनसक्छन ?भविष्यमा हस्तान्तरण गरिने आयोजनाहरुमा दिगोपना ल्याउन के के गर्नुपर्ला ?सुझाव प्रस्तुत गर्नुहोस् |२०

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२०७६/९/११गते

समय –: १ घण्टा ३० मिनेट पूर्णाक –: ५०

पत्र –: प्रथम , खण्ड) ख (बिषय –: सेवा सम्बन्धि सामान्य बिषय

तलका प्रश्नको उतर छुटाछुटै उतरपुस्तिकामा लेख्नुपर्नेछ अन्यथा उतर पुस्तिका रद्द हुनेछ ।

1. Describe the principle components of national water resources strategy, 2002. How would you relate it with the irrigation regulation, 2056? 10
2. सिंचाई नीति, २०६० र नियमावली ,२०५६ मा गरिएका संशोधन बमोजिम निजि क्षेत्रको सहभागिता बान्छित रुपमा बढन नसक्नुको कारण पहिल्याउदै अर्थपूर्ण सहभागिता अभिबृधि गर्न नेपाल सरकारले के कस्ता कदम चाल्नुपर्ला? विश्लेषण गर्नुहोस् |
3. वातावरण संरक्षण नियमावली ,२०५४ मा वातावरणीय प्रभाव परिक्षण)EIA (सम्बन्धमा भएको व्यवस्था बारे उल्लेख गर्नुहोस् |उर्जा, जलस्रोत तथा सिंचाई मन्त्रालय/विभाग बाट प्रभावकारी अनुगमन कार्यको लागि के कस्तो संयन्त्र आवश्यक देख्नुहुन्छ ?प्रकाश पार्नुहोस् |१०
4. विभिन्न आयोजनाहरु समयमा जग्गा प्राप्त हुन् नसकेको कारण ढिलो गरि सम्पन्न भैरहेको स्थितिमा जग्गा प्राप्ति ऐन ,२०३४ को सान्दर्भिकता र भएका व्यवस्थाको पर्याप्तताबारे विश्लेषण गर्नुहोस् |जग्गा प्राप्त गर्ने काममा सरलीकरण गर्न उक्त ऐनमा गर्नुपर्ने समसामयिक परिमार्जन बारे उल्लेख गर्नुहोस् |५+५
5. What are the aims and objectives of Nepal Engineering Council? Are the objectives achievement? List the actions needed to make the council more effective. 10

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१. जलस्रोत ऐन ,२०४९ अनुसार जलस्रोतको विभिन्न उपयोगको लागि कस्तो प्राथमिकता तोकिएको छ? यसमा व्यवहारिक रुपमा भएको उपयोगको अवस्थाबारे विवेचना गर्नुहोस् |५+५

२. सिंचाई विकास सम्बन्धि मुख्य मुख्य चुनौतिहरु के के हुन् ?जलवायु परिवर्तनको बर्तमान अवस्थामा निर्माण भैसकेको सिंचाई परियोजनालाई पूर्ण क्षमताम संचालन गर्न आवशयक उपायहरु सुझाउनुहोस् |आगामी सिंचाई आयोजनाहरुमा जलबायु परिवर्तन बाट हुने असरलाई न्यून गर्ने तरिकाहरु उल्लेख गर्नुहोस् |३+४+३

३. वातावरण संरक्षण ऐन ,२०५३ तथा नियमावली ,२०५४ अनुसार सिंचाई क्षेत्रमा गर्नुपर्ने वातावरणीय प्रभाव मूल्यांकन)EIA (का प्रावधानहरु उल्लेख गर्दै ति वातावरणीय प्रतिवेदनहरुको प्रभावकारिता बारे चर्चा गर्नुहोस् |

४. नेपाल तथा भारत बीच भएको गण्डक सम्झौता तथा कोशी सम्झौतामा आधारभूत रुपमा के के भिन्नताहरु छन् ?यी सम्झौताहरुमा दुई देश बीच गरिएको पानीको बाँडफाड नेपालको हित/अहितमा भएका कसरी मूल्यांकन गर्नुहुन्छ ?स्पष्ट पार्नुहोस् |४+६

५. सार्वजनिक खरिदको विद्यमान कानूनी प्रक्रिया झन्झटिलो र अत्यधिक समय लाग्ने भनेर आलोचना हुने गरेको सन्दर्भमा यसको प्रक्रियागत सुधारका लागि कस्तो सुझाव उपयुक्त हुन्छ ?स्पष्ट गर्नुहोस् |१०

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तलका प्रश्नको उतर छुटाछुटै उतरपुस्तिकामा लेख्नुपर्नेछ अन्यथा उतर पुस्तिका रद्द हुनेछ ।

१. नेपालको सिंचाई विकास सम्बन्धि चुनौतिहरु बारे समिक्षा गर्नुहोस् ।१०
२. निजामती सेवा सम्बन्धि प्रचलित कानुनमा भएको कर्मचारीहरुको बढुवा सम्बन्धि व्यवस्थाले कर्मचारीहरुको बढुवामा के कस्तो प्रभाव पारेको छ ?उल्लेख गर्दै यस बारे कुनै समस्या भए सोको समाधानको लागि उपयुक्त सुझाव प्रस्तुत गर्नुहोस् ।१०
३. वातावरणीय प्रभाव मूल्यांकन भन्नाले के बुझ्नुहुन्छ ?वातावरण ऐन ,२०५३ र नियमावली ,२०५४ अन्तर्गत के कस्तो प्रस्ताव के कस्तो कार्यविधि हरु पुरा गरी वातावरणीय प्रभ मूल्यांकन गर्नु पर्दछ ? सक्षिप्त रुपमा चर्चा गर्नुहोस् ।
४. सिंचाई र नदि नियाँत्रन्को कार्य छुटाछुटै निकाय बाट संचालन गरिने वर्तमान संरचना का कारण कतिपय अवरोधहरु सिर्जना भइ आयोजना कार्यान्वयनमा समस्या देखिएका छन् ।यस्ता निकायहरु बीच समन्वय स्थापना गरि आयोजनाहरु प्रभावकारी ढंगले संचालन गर्न के गर्नुपर्ला ?आफ्नो धारणा प्रस्तुत गर्नुहोस् ।१०
५. एक देश बाट आर्को देशमा बग्ने नदीहरुलाई सिंचाई तथ् और्जको लागि विकास गर्ने सिलसिलामा upperriparian तथा lowerriparian सम्बन्धि पानीको अधिकार सम्बन्धि राष्ट्रिय तथा अन्तर्राष्ट्रिय सन्धि महासन्धि हरुलाई नेपालको सन्दर्भमा बिश्लेषणात्मक टिप्पणी गर्नुहोस् ।१०

नेपाल इन्जिनियरिङ सेवा, सिभिल समूह ,इरिगेशन उप समूह ,राजपत्रांकित प्रथम श्रेणी ,सहसचिव वा सो सरह प्राबिधिक पदको प्रतियोगितात्मक लिखित परिक्षा

२०७१/८/१७ गते

समय –:१ घण्टा ३० मिनेट पूर्णाक –:५०

पत्र –:प्रथम ,खण्ड)ख (बिषय –:सेवा सम्बन्धि सामान्य बिषय

तलका प्रश्नको उतर छुटाछुटै उतरपुस्तिकामा लेख्नुपर्नेछ अन्यथा उतर पुस्तिका रद्द हुनेछ ।

१. नेपालको राष्ट्रिय जलस्रोत रणनीतिले परिकल्पना गरेका आधारभूत कुराहरु के के हुन् ?सविस्तार बर्णन गर्नुहोस् ।१०
२. नेपालमा संस्थागत रुपमा सिंचाईको विकस गर्न विद्यमान् सिंचाई नीति र सिंचाई नियमावलीमा के कस्ता व्यवस्था हरु उल्लेख गरिएका छन् ?तपाईको विचारमा यी व्यवस्थाहरु पर्याप्त छन् वा छैनन् ? विश्लेषणात्मक विवेचना गर्नुहोस् ।
३. कृषि उत्पादकत्व वृद्धिको सन्दर्भमा भूमि व्यवस्थापन र सिंचाई व्यवस्थापन लाई कसरी समन्बयात्मक रुपमा लैजानुपर्ने देख्नुहुन्छ ?विवेचना गर्नुहोस् ।
४. सार्वजनिक निर्माण सम्बन्धि काम गराउंदा लागत अनुमान तयार गर्नुपर्ने र स्वीकृति दिनुपर्ने समबन्धमा सार्वजनिक खरिद ऐन ,२०६३ एव नियमावली २०६४ मा के कस्ता व्यवस्था गरिएको छ ?सक्षिप्तमा उल्लेख गरी यसको औचित्य स्पष्ट गर्नुहोस् ।१०
५. सिंचाई तथा उर्जा विकासको कार्य केही बर्ष अघि सम्म एकै मन्त्रालय "जलस्रोत मन्त्रालय "अन्तर्गत रहेका थिए ।तर विभिन्न कारणले उक्त मन्त्रालय दुइवटा मन्त्रालयमा विभाजित हुन् गएको छ – सिंचाई मन्त्रालय र उर्जा मन्त्रलय ।

   नेपालमा अव उप्रान्त बहुउदेशीय आयोजनाहरु – जस अन्तर्गत सिंचाई, उर्जा ,नदि नियन्त्रण, नाभिगेशन आदि विकासका काम पर्दछन – र यसको निर्माण अगाडी बढाउने योजना तर्जुमा गरिएको छ ।यसरी

वर्तमान संरचनाका कारण कतिपय अवरोध हरु सिर्जना भई आयोजना कार्यान्वयनमा समस्या देखिएका छन् |यस्ता निकाय बीच समन्वय स्थापना गरि आयोजनाहरु प्रभावकारी ढंग ले संचालन गर्न के गर्नुपर्ला ?आफ्नो धारणा प्रस्तुत गर्नुहोस् |१०

नेपाल इन्जिनियरिङ सेवा, सिभिल समूह ,इरिगेशन उप समूह ,राजपत्रांकित दितीय श्रेणी ,उपसचिव वा सो सरह प्राबिधिक पदको प्रतियोगितात्मक लिखित परिक्षा

२०७६/१०/१७ गते

समय –: ३ घण्टा पूर्णाक –: १००

पत्र –: दित्तिय बिषय –: सेवा सम्बन्धि प्राविधिक बिषय

तलका प्रश्नको उतर छुटाछुटै उतरपुस्तिकामा लेख्नुपर्नेछ अन्यथा उतर पुस्तिका रद्द हुनेछ ।

1. What are the elements of participatory approach at different stages of an irrigation project like planning, construction, operation and maintenance? Discuss the advantages and disadvantages of participatory approach in context of Nepal? 15
2. What types of new irrigation projects have to carry out environmental impact assessment as per the environmental protection act and regulation? Analyze rational of EIA in the irrigation sector. Do you suggest any reform needed in the existing provision of EIA? 8+3+4
3. State the importance of ground water exploration in Nepal. Name the various direct and indirect methods of exploration and discuss briefly the relative's merit and demerits of the two. 5+10
4. Enumerate the advantages of permeable spurs against impermeable one. Drawing neat sketches explain how various types of permeable spurs are useful in Nepalese context. 5+10
5. Define water conveyance efficiency and water application efficiency of irrigation systems. Describe briefly the method of calculation of water requirement for crops in irrigation projects on the basis of planning and design strengthening project (PDSP) design manual. 20
6. Discuss the major problems and issues of irrigation development in Nepal. Make a critical review of national policies. Give your suggestion for enhancing effective in implementation. How should your suggestions be implemented and monitored? 6+6+4+4

नेपाल इन्जिनियरिङ सेवा, सिभिल समूह ,इरिगेशन उप समूह ,राजपत्रांकित दितीय श्रेणी ,उपसचिव वा सो सरह प्राबिधिक पदको प्रतियोगितात्मक लिखित परिक्षा

२०७४/११/११ गते

समय –: ३ घण्टा पूर्णाक –: १००

पत्र –: दित्तिय बिषय –: सेवा सम्बन्धि प्राविधिक बिषय

तलका प्रश्नको उतर छुटाछुटै उतरपुस्तिकामा लेख्नुपर्नेछ अन्यथा उतर पुस्तिका रद्द हुनेछ ।

1. Classify the irrigation systems in Nepal according to their management and performance. Discuss their pros and cons and explain their suitability for particular situations. 15
2. How do you ensure that the tube well construction has been satisfactory? Describe any test you would conduct for this purpose. 15
3. It is normal practice in DWIDM to construct a series of gabion spurs to divert the flow away from the outer bank of the lateral shifting river forming a sharp bend. In many situations such installation is either not functional or in damaged state. Elaborate the reasons behind such situation. With justification propose the remedial measures. 15
4. What are the problems in mustering equity for irrigation project in Nepal? Suggest a model for generating public investment in irrigation projects. 15
5. बाढी ,पहिरो, नदि कटान, डुवान तथा भू-स्खलन समस्याको समधको लागि जिम्मेवारी निकाय प्रभावकारी हुन् नसक्दा तत् सम्बन्धि योजना तर्जुमा, छनौट तथा कार्यान्वयन चरण समेत कमजोर भएको महशुस भएको छ ।यस सम्बन्धमा आफ्नो विचार सहित समस्याको विश्लेषण गर्दै समाधानका उपाय स्पष्ट पार्नुहोस् ।२०
6. List the common problems faced by the existing head works of irrigation schemes in Nepal. Explain the causes behind these problems and suggest suitable measures to avoid these problems and suggest suitable measures to avoid them in designing head works in new irrigation schemes.20

नेपाल इन्जिनियरिङ सेवा, सिभिल समूह ,इरिगेशन उप समूह ,राजपत्रांकित दितीय श्रेणी ,उपसचिव वा सो सरह प्राबिधिक पदको प्रतियोगितात्मक लिखित परिक्षा

२०७२/०९/१३ गते

समय –: ३ घण्टा पूर्णाक –: १००

पत्र –: दित्तिय बिषय –: सेवा सम्बन्धि प्राविधिक बिषय

तलका प्रश्नको उतर छुटाछुटै उतरपुस्तिकामा लेख्नुपर्नेछ अन्यथा उतर पुस्तिका रद्द हुनेछ ।

1. A) What is the importance of drainage in irrigated agricultural land? Provide neat sketch of drainage networks. 3+3

   B) Calculate the discharge carrying capacity of an irrigation canal (Trapezoidal section) with Manning's n=0.015, longitudinal grade S=1/10000 and side slope of 1 (H): 0.75 (V). Assume bed width and depth of flow as 4m and 1.5 m respectively. 9
2. Explain how can you develop a flood warning system for Bagmati River in Terai with existing available information's and database. 15
3. Describe with appropriate details different stages in the participatory approach in irrigation project planning and implementation of farmer's hill irrigation projects. 15
4. River flowing through Nepal carries large quantity of sediments, both suspended and rolling along with its water. What steps should be taken to minimize sedimentation on agriculture land?
5. Discuss pros and cons of People's Embankment Pragramme (PEP). Suggest suitable remedies to bridge the existing gaps for suitable reduction of inundation and river bank cutting in Nepal through such programs. 20
6. Describe various types of irrigation systems with respect to their management. Critically examine the joint management concept including its merits and demerits. Suitable your answer with examples from Nepal. 20

INTRODUCTION

IRRIGATION

Three basic requirements of agricultural production are soil, seed, and water. In addition, fertilizers, insecticides, sunshine, suitable atmospheric temperature, and human labour are also needed. Of all these, water appears to be the most important requirement of agricultural production. The application of water to soil is essential for plant growth and it serves thefollowing functions (1):

It supplies moisture to the soil essential for the germination of seeds, and chemical and bacterial processes during plant growth.

It cools the soil and the surroundings thus making the environment more favourable for plant growth.

It washes out or dilutes salts in the soil.

It softens clods and thus helps in tillage operations.

It enables application of fertilizers.

It reduces the adverse effects of frost on crops.

It ensures crop success against short-duration droughts.

In several parts of the world, the moisture available in the root-zone soil, either from rain or from underground waters, may not be sufficient for the requirements of the plant life. This deficiency may be either for the entire crop season or for only part of the crop season. For optimum plant growth, therefore, it becomes necessary to make up the deficiency by adding water to the root-zone soil. This artificial application of water to land for supplementing the naturally available moisture in the root-zone soil for the purpose of agricultural production is **termed irrigation.**

Irrigation water delivered into the soil is always more than the requirement of the crop for building plant tissues, evaporation, and transpiration. In some cases the soil may be naturally saturated with water or has more water than is required for healthy growth of the plant. This excess water is as harmful to the growth of the plant as lack of water during critical stages of the plant life. This excess water can be naturally disposed of only if the natural drainage facilities exist in or around the irrigated area. In the absence of natural drainage, the excess water has to be removed artificially. The artificial removal of the excess water is termed drainage which, in general, is complementary to irrigation.

To keep the optimum content of water in soil, irrigation supplies water to the land where water is deficient and drainage withdraws water from the land where water is in excess. The object of providing irrigation and drainage is to assist nature in maintaining moisture in the root-zone soil within the range required for maximum agricultural production. Usefulness and importance of irrigation can be appreciated by the fact that without irrigation, it would have been impossible for India to have become self-sufficient in food with such huge population to feed. Primary source of prosperity in Punjab is irrigation. Irrigation from the Nile is the source of food, life, and prosperity in Egypt. Similarly, without drainage, large parts of the Netherlands and the coastal regions of several countries would always be under water.

Irrigation schemes can be broadly grouped into two main categories: (i) surface water irrigation schemes, and (ii) ground water irrigation schemes. The former use diversion and storage methods and obtain their supplies from rivers. Ground water irrigation schemes use open wells, and deep and shallow tube wells to lift water from the water-bearing strata below the earth's surface. The choice of an irrigation scheme depends on several factors, such as surface topography, rainfall characteristics, type of source available, subsoil profile, etc. One should, however, always plan to use surface and ground waters together to derive maximum benefits. Such use is termed conjunctive use of surface and ground waters.

IMPACT OF IRRIGATION ON HUMAN ENVIRONMENT

The main impact of irrigation is in terms of the increased agricultural yield which, in turn, affects social, cultural, economic, political and other aspects of human.

Impact of irrigation on human environment

Impact	Positive	Negative
Engineering	Improvement of the water regime of irrigated soils. Improvement of the micro climate. Possibility provided for waste water use and disposal. Retention of water in reservoirs and possible multipurpose use thereof.	Danger of water logging and salination of soils, rise in ground water table. Changing properties of water in reservoirs. Deforestation of area which is to be irrigated and with it a change of the water regime in the area. Reservoir bank abrasion.
Health	Securing increased agricultural production and thus improving the nutrition of the population. Recreation facilities in irrigation canals and reservoirs.	Possible spread of diseases ensuing from certain types of surface irrigation. Danger of the pollution of water resources by return runoff from irrigation. Possible infection by waste water irrigation, new diseases caused by retention of water in large reservoir.
Social and Cultural	Culturing the area. Increasing the social and cultural level of the population. Tourist interest in the area of the newly-built reservoir.	Colonization of the irrigated area. Displacement of population from retention area. Necessity of protecting cultural monuments in inundated areas.
Aesthetic	New man-made lakes in the area.	Project's architecture may not blend

12

		with the area.
Political	Increased self-sufficiency in food, thus lesser dependence on other countries.	

Reliable and adequate irrigation is known to raise the employment. According to some field studies, increases in working days per hectare with irrigation compared with rainfed conditions have been 61 percent as high as 150 percent. As a result, production as well as incomes is generally stable at higher levels.

Due to the assured employment and higher incomes spaced over the entire year, there is added security against impoverishment. Therefore, the need for having dependent relationships with moneylenders and employers as well as the dangers of having to dispose of assets like land to buy food or meet debts is much less.

Another beneficial aspect of irrigation is that it stops exodus and attracts people to the region. Therefore, hardships associated with split families are avoided and a more stable and settled family life results.

Irrigation influences the quality of life. One major effect is the increase in prosperity which must improve the nutrition intake and resistance of the people against disease. The prosperity, in its wake, does bring some evils such as dowry, drug habits, etc. But, these evils can be eradicated through education and social welfare programs. In addition to the above- mentioned gains in agricultural production and livelihood of rural population, irrigation also provides protection against famine and increases the quality of agricultural yields. Other secondary benefits of irrigation projects, such as hydroelectric power generation, use of canals for inland navigation, domestic water supply, and improvement in communication systems also affect the human environment in a favourable manner.

There can, however, be adverse effects too. The adverse effects are mainly in the form of waterborne and water-related diseases and waterlogged saline lands.

The performance of major and medium irrigation schemes has following deficiencies.

- Need for modernization of the pre-plan and early-Plan systems to provide water at the outlet delivery points to farmers at the right time and in the right quantity.
- Lack of adequate drainage like Sunsari Morang Irrigation, Bagmati Irrigation resulting in water logging conditions due to excess water used in irrigating crops as well as due to soil characteristics.
- The absence of a distribution system within the outlet and the non-introduction of rotational distribution of water to the farmers.
- Inadequate attention to land consolidation, levelling and all other aspects which can promote a better on-farm management of water.

13

- Lack of anticipatory research on optimum water use, particularly in black soils withconsiderable moisture retention capacity.
- Lack of suitable infrastructure and extension services.
- Poor coordination between the concerned Government organizations in the commandareas.
- Heavy siltation like Sunsari Morang Irrigation, Bagmati Irrigation
- Inappropriate Location of Barrage/Weir or Head works
- Inappropriate and untimely maintenance works of irrigation structure like Sikta irrigation, Gankadak irrigation
- Distraction in Agricultural professional mostly by youth people
- Haphazardly land use pattern or built up area development and migration
- Wastage and misuse of water in upper side of canal

COMMAND AREA DEVELOPMENT

The irrigation potential created by the construction of a large number of major and medium irrigation projects has more than doubled since independence. However, the available irrigation potential has always been under-utilized and the optimum benefits by way of increased production have not been fully realised. Several studies have been made to analyse the reasons for inefficient and continued under-utilisation of available irrigation potential and unsatisfactory increase in agricultural production in irrigated areas. Agriculture sector recommended an integrated command area development programme for optimising benefits from available irrigation potential. The objectives of the programme were as follows :

1. Increasing the area of irrigated land by proper land development and water management.
2. Optimising yields by adopting the best cropping pattern consistent with the availability of water, soil, and other local conditions.
3. Bringing water to the farmer's field rather than only to the outlets and thus assuring equitable distribution of water and adequate supply to tailenders.
4. Avoiding wastage and misuse of water.
5. Optimising the use of scarce land and water resources, including ground water where available, in conjunction with necessary inputs and infrastructure.

The command area development programme is a series of coordinated measures for optimising the benefits from the irrigated agriculture. Some of these measures are :

1. Scientific crop planning suited to local soil and climatic conditions.
2. Consolidation of holdings and levelling/shaping of lands.
3. Provision of field channels to ensure equitable distribution of water to the

farmer's field.

4. Ensuring the supply of other inputs (good quality seeds, fertilisers, etc.).
5. Construction of rural roads, markets, storages, and other infrastructural facilities inthe command areas of irrigation projects.

Major programme included the following components.

1. Modernisation and efficient operation of the irrigation system.
2. Development of a main drainage system beyond the farmer's blocks.
3. Construction of field channels and field drains.
4. Land shaping/levelling and consolidation of holdings.
5. Lining of field channels/watercourses.
6. Exploitation of ground water and installation of tubewells.
7. Adoption and enforcement of a suitable cropping pattern.
8. Enforcement of an appropriate rostering system on irrigation.
9. Preparation of a plan for the supply of key inputs like credit, seeds, fertilisers, pesti-cides, and implements.
10. Making arrangements for timely and adequate supply of various inputs.
11. Strengthening of existing extension, training, and demonstration organisations.

Agriculture emphasized the need for development of land in the command area in an integrated manner comprising the following actions:

1. Layout of plots and of common facilities like watercourses, field channels, drains and farm roads.
2. Consolidation of farmers' scattered plots into one or two operational holdings.
3. Construction of watercourses and field channels.
4. Construction of field drains where necessary and linking them with connecting drains
5. Provision of farm roads.
6. Land formation to suitable slopes.

PLANNING OF IRRIGATION PROJECTS

Agricultural establishments capable of applying controlled amounts of water to lands to producecrops are termed irrigation projects. These projects mainly consist of engineering (or hydraulic) structures which collect, convey, and deliver water to areas on which crops are grown. Irrigation projects may range from a small farm unit to those serving extensive areas of millions of hectares. A small irrigation project may consist of a low diversion weir or an inexpensive pumping plant along with small ditches (channels) and some minor control structures. A largeirrigation project includes a large storage reservoir, a huge dam, hundreds of kilometres of canals, branches and distributaries, control structures, and other works. Assuming all other factors (such as enlightened and experienced farmers, availability of good seeds, etc.) reasonably favourable, the following can be listed as conditions essential for the success of any irrigationproject.

1. Suitability of land (with respect to its soil, topography and drainage features) for

continued agricultural production,

2. Favourable climatic conditions for proper growth and yield of the crops,
3. Adequate and economic supply of suitable quality of water, and
4. Good site conditions for the safe construction and uninterrupted operations of the engineering works.

During the last decades, many large irrigation projects are under construction as multipurpose projects. Such projects serve more than one purpose of irrigation or power generation. Most of the irrigation projects divert stream flow into a canal system which carries water to the cropland by gravity and, hence, are called gravity projects. In pumping projects, water is obtained by pumping but delivered through a gravity system. A gravity type irrigation project mainly includes the following works:

1. Storage (or intake) and diversion works,
2. Conveyance and distribution channels,

3. Conveyance, control, and other hydraulic structures,
4. Farm distribution, and
5. Drainage works.

Development of an Irrigation Project

A small irrigation project can be developed in a relatively short time. Farmers having land suitable for agriculture and a source of adequate water supply can plan their own irrigation system, secure necessary finance from banks or other agencies, and get the engineering works constructed without any delay. On the other hand, development of a large irrigation project is more complicated and time-consuming. Complexity and the time required for completion of a large project increase with the size of the project. This is due to the organizational, legal, financial administrative, environmental, and engineering problems all of which must be given detailed consideration prior to the construction of the irrigation works. The principal stages of a large irrigation project are: (i) the promotional stage, (ii) the planning stage, (iii) the construction stage, and (iv) the settlement stage. The planning stage itself consists of three sub stages: (i) preliminary planning including feasibility studies, (ii) detailed planning of water and land use, and (iii) the design of irrigation structures and canals.

Engineering activities are needed during all stages (including operation and maintenance) of development of an irrigation project. However, the planning and construction stages require most intensive engineering activities. A large irrigation project may take 10–30 years for completion dependingupon the size of the project.

Feasibility of an Irrigation Project

A proposed irrigation project is considered feasible only when the total estimated benefits of the project exceed its total estimated cost. However, from the farmer's viewpoint, an irrigation project is feasible only if his annual returns (after completion of the project) exceed his annual costs by sufficient amount. The feasibility of an irrigation project is determined on the basis of preliminary estimates of area of land suitable for irrigation, water requirements, available water supplies, productivity of irrigated land, and required engineering works.

Planning of an Irrigation Project

Once the project is considered feasible, the process of planning starts. Sufficient planning of all aspects (organisational, technical, agricultural, legal, environmental, and financial) is essential in all irrigation projects. The process of planning of an irrigation project can be divided into the following two stages:

Preliminary planning, and 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 theexpertise 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 return etc

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 instudies 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 irrigationstructures 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 allsuch 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.

Environmental Check-List for Irrigation and Water Resource Projects

The term environment includes the earth resources of land, water, air, vegetation, and man- made structures. The relationship between organisms (i.e., plant, animal and human) andtheir environment is termed ecology. All water resource projects, whether for irrigation or for Hydro-electric power or for flood control or for water supply, are constructed for the well-being of human beings and have definite impact on the surrounding ecosystems and environment. It is, however, unfortunate that some of the environmentalists get unreasonably influenced by the subtle propaganda against the development of water resources in India by the people of the developed nations who would not like the people of India to be able to reach near the level of living style of the people of the developed countries. These people oppose development of water resources in Nepal on environmental considerations without appreciating the needs of Nepal and the fact that Nepal has utilized little per cent of its utilizable potential. As a result, the per capita consumption of electric power and all other human needs is much lower than that in the developed countries. Region-wise, Nepal is already a water-short country and faces acute water problems in almost the entire country. This will continue to be so till India controls the increase in its population and harnesses its entire monsoon and redistributes itspatially and temporally. The mooted proposal of interlinking of rivers in the country envisages inter-basin transfers of surplus water to meet the water needs of the water-shortregions of the country. Such developmental works do cost a fortune in terms of money and environmental impacts. However, if the benefits (monetary as well as environmental) exceed the cost,(both monetary and environmental), the work should be considered justifiable. The decision of water resources development should be based upon analyzing the future scenario 'with' and 'without' the proposed development. Therefore, the developmental activities cannot be stopped on environmental considerations alone. It should, however, be appreciated that both developmental activities and an intact environment are equally important for sustained well-being of human beings. Therefore, the water resources projects must be developed such that they minimize environmental disturbances and maintain ecological balance while meeting the demands of man.

The complexity of environmental processes seldom permits accurate prediction of the full spectrum of changes in the environment brought about by any particular human activity. Many countries, including India, have now made it a statutory requirement for environmental impact assessment (EIA) of all new projects within specified category. Water resources projects are included in this category and are approved only after favourable report of EIA studies. The statutory EIA authorities usually concentrate on negative aspects of environmental changes. This results in conflict between the EIA authorities and project planners. Since EIA requires detailed information, it is usually undertaken at the final stage of the project planning when changes in the project to mitigate adverse effects on environment are difficult and costly.

The environmental check-list provides a comprehensive guide to the areas of environmental concern which should be considered in the planning, design, operation, and management of irrigation, drainage, and flood control projects. This check-list provides a tool which will enable planners concerned with irrigation and drainage development to appreciate the environmental changes which such projects may bring about so that adverse effects can be identified and, if possible, avoided or, at least, controlledand positive effects enhanced.

Hydrological Change

Low flow regime. Is the low flow regime of the river substantially changed by the Project and its dams (by more than ± 20% in low flow periods)? If so, does this change benefit or impair aquatic ecosystems, existing or potential downstream abstractions, hydropower navigation or recreational uses?

Flood regime. Is the flood regime of the river (peak discharge and stage, speed of flood waves, flood super-position with joining rivers, duration or extent of floodplain inundations downstream) substantially changed by the Project as well as the result of changes in abstractions, retention storage, reservoir releases, flood protection works, new road/rail routes, river training or surface drainage works? If so, does this change benefit or impair aquatic and flood-affected ecosystems, lead to an increase or decrease in flood damage or change land use restriction outside the project?

Operation of dams. Can modifications to the operation of any storage or flood retention reservoir(s) compensate for any adverse impacts associated with changes in flow regime, whilst minimizing the losses to the Project and other users? Possible modifications affecting water quality downstream, saline intrusion, the sediment regime of channels, the ecology of affected area, amenity values, disease transmission or aquatic weed growth should be considered. (A separate environmental assessment of large reservoir(s) may be required).

Fall of water table. Does the Project cause a fall of the water table (from groundwater abstractions, reduced infiltration due to river training, drainage or flood protectionworks)? If so, does this fall lead to increased potential for groundwater recharge (from seasonal rainfall) and improved conditions for land use; or lead to depletion of the groundwater system, affecting wells, springs, river flows and wetlands?

Rise of water table. Does the Project cause a rise of the water table (from increased infiltration or seepage from irrigation, seepage from reservoirs and canals or increased floodplain inundation)? If so, does this rise lead to improved yield of wells and springs and improved capillary rise into the root zone; or lead to water logging of agricultural or other land in the Project area or vicinity?

Organic and Inorganic Pollution

Solute dispersion. Are the Project and its dams leading to changes in the concentrations of organic or inorganic solutes in the surface water due to changes to the pattern of water abstraction and reuse in the basin or flow regulation? If so, do the changes benefit or impair biological communities or domestic, agricultural orindustrial water users in the basin?
Toxic Substances. Are significant levels of toxic substances accumulating or being introduced, mobilised and transmitted due to the construction and operation of the Project and its dams,

or are levels being reduced? Substances such as pesticides, herbicides, hydrogen sulphide, oil derivatives, boron, selenium and heavy metals in irrigation supplies or surface, drainage and ground waters should be considered.

Organic pollution. Are nutrients, organic compounds and pathogens being reduced or introduced and concentrated, due to the Project, its dams and its associated domestic settlements? If so, does the change result in a reduction or increase in environmental and water use problems in the Project area or downstream (in rivers, canals, reservoirs, end lakes, evaporation wet lands, depressions, deltas, estuary regions) or in the groundwater?

Anaerobic effects. Is the Project reducing or creating anaerobic conditions or eutrophication in any impoundments, natural lakes, pools or wetlands due to changed input or accumulation of fertilisers, other nutrients and organic matter or due to changed water quality resulting from dams, river abstractions and drainage flows?

Gas Emissions. Is the Project, either directly or through associated industrial processing, causing decreased or increased gas emissions which contribute to air pollution (O_3, SO_3, H_2S, NO_x, NH_4, etc.) or the greenhouse effect (CO_2, CH_4, NO_x, etc.)?

Soil Properties and Salinity Effects

Soil salinity. Is the Project leading to progressive accumulation of salts in the soils of the project area or the vicinity because of prevailing high salt content in, the soil, the groundwater, or the surface water; or can a progressive leaching effect be expected?

Soil properties. Is the project leading to changes in soil characteristics within the Project area or the vicinity due to such activities as irrigation, the application of fertilisers or other chemicals, cultivation practices or dewatering through drainage? Changes which can improve or impair soil structure, workability, permeability, fertility associated with nutrient changes, humus content, pH, acid sulphate or hard pan formation or available water capacity should be considered.

Saline groundwater. Are changes to the rates of seepage, percolation or leaching from the Project and its dams increasing or decreasing the concentrations of chlorides, nitrates or other salts in the groundwater?

Saline drainage. Are changes to the concentrations of chlorides, nitrates or other salts in the runoff or drainage water from the Project area in danger of affecting biological communities or existing or potential downstream users (particularly during low flow conditions)?

Saline intrusion. Are the Project and its dams leading to changes in saline water (sea water) intrusion into the estuary or into groundwater due to changes in low flow, groundwater use, dredging or river training? If so, are the changes likely to affect biological communities and water users in the Project vicinity and other areas?
Erosion and Sedimentation

Local erosion. Is increased or decreased soil loss or gully erosion being caused within or close to the Project area by changes in land gradient and vegetative cover, by irrigation and cultivation practice, from banks of canals, roads and dams, from areas of cut and fill or due to storm drainage provision?

Hinterland effect. Are the Project and its dams leading to changes in natural vegetation, land productivity and erosion through changes in population density, animal husbandry, dryland farming practices, forest cover, soil conservation measures, infrastructure development and economic activities in the upper catchment and in the region surrounding the Project?

River morphology. Is the regime of the river(s) changed by the Project and its dams through changes in the quantity or seasonal distribution of flows and flood peaks in the river(s), the abstraction of clear water, changes in sediment yield (caused by 4.1 and 4.2), the trapping of sediment in reservoirs or the flushing of sediment control structures? If so, do these changes benefit or impair aquatic ecosystems or existing or potential users downstream?

Channel structures. Is scouring, aggradation or bank erosion in the river(s) endangering the Project's river headworks, offtake structures, weirs or pump inlets, its canal network, drainage or flood protection works, the free flow of its drainage system or structures and developments downstream? Consider effects associated with change as well as those caused by other existing and planned upstream developments.

Sedimentation. Are the changes noted in causing increased or decreased sediment deposition in irrigation or drainage canals, hydraulic structures, storage reservoirs or on cultivated land, either via the irrigation system or the river(s)? If so, do these changes benefit or impair soil fertility, Project operation, land cultivation or the capacity and operation of reservoirs?

Estuary erosion. Are the Project and its dams leading to changes in the hydrological or sediment regimes of the river which can affect delta formation or estuary and coastal erosion? If so, do these changes benefit or impair aquatic ecosystems (estuarine or marine), local habitation, navigation or other uses of the estuary?

Biological and Ecological Changes

Is the Project, its dams or its associated infrastructure causing substantial and permanent changes (positive or negative) within the habitats listed in ?
in the natural ecology (habitat, vegetation, terrestrial animals, birds, fish and other aquatic animals and plants), in areas of special scientific interest, or in biological diversity Include the likely ecological benefit of any new or modified habitats created and of any protective or mitigatory measures adopted (such as nature reserves and compensatory forests).

Project lands. The lands within the project area.

Water bodies. Newly created, altered or natural channels, reservoirs, lakes and rivers.

Surrounding area. All terrestrial areas influenced by the Project works and its associated domestic settlements and hinterland effects.

Valleys and shores. River and canal banks, lake, reservoir and sea shores and the offshore marine environment.

Wetlands and plains. Floodplains or permanent wetlands including deltas and coastal swamps.

Rare species. Is the existence of any rare, endangered or protected species in the region enhanced or threatened by the changes?

Animal migration. Does the Project, its dams or new road/rail routes affect the migration patterns of wild animals, birds or fish? Make allowance for the compensatory effect or any additional provision within the Project (canal crossings, fish passes, spawning locations, resting or watering places, shade, considerate operation).

Natural industry. Are commercial or subsistence activities depending on the natural terrestrial and aquatic environment benefited or adversely affected by the Project through ecological changes or changes in human access? Changes affecting such activities as fisheries, harvesting from natural vegetation, timber, game hunting or viewing and honey production should be considered.

Socio-Economic Impacts

Population change. Is the Project causing significant demographic changes in the Project area or vicinity which may affect social harmony? Changes to population size/density and demographic/ethnic composition should be considered.

Income and amenity. Is the Project introducing significant economic/political changes which can increase or decrease social harmony and individual well-being? Changes in the general levels of employment and income, in the provision of local infrastructure and amenities, in the relative distribution of income, property values and Project benefits (including access to irrigation water) and in the demand for labour and skills (particularly in relation to family/political hierarchy and different sexes and social groups) should be considered.

Human migration. Has adequate provision been made for any temporary or migratory population influx to avoid social deprivation, hardship or conflicts within these groups or between the permanent and temporary groups? Human migration arising both from the demand for skills/labour during construction and from the requirements for seasonal agricultural labour should be considered.

Resettlement. Has adequate provision been made for the resettlement, livelihood and integration of any people displaced by the Project and its dams or losing land, grazing or other means of income due to the Project? Also, has adequate provision been made for the subsistence farming needs of people settled on or associated with the Project?
Women's role. Does the Project change the status and role of women (positively or negatively) in relation to social standing, work load, access to income and heritage and material rights?

Minority groups. Are the Project and its dams causing changes to the lifestyle, livelihoods or habitation of any social groups (particularly minority groups) leading to major conflicts with, or changes to their traditional behaviour, social organisation or cultural and religious practices?

Sites of value. Is access improved or hampered to places of aesthetic and scenic beauty, sites of historical and religious significance or mineral and palaeontological resources? Also, are any such sites being destroyed by the Project?

Regional effects. Are the economic, infrastructural, social and demographic changes associated with the Project likely to enhance, restrict or lead to unbalanced regional development? Also, has adequate provision been made for new transport, marketing and processing needs associated with the Project?

User involvement. Has there been adequate user and public participation in project planning, implementation and operation to ensure Project success and reduce future conflicts? The potential for incorporating within the Project existing systems of land tenure, traditional irrigation, and existing organisational and sociological structures and for the provision of new or extended facilities for credit, marketing, agricultural extension and training should be considered.

Recreation. Are the Project and its dams creating new recreational possibilities (fishing, hunting, sailing, canoeing, swimming, scenic walks, etc.) and are existing facilities impaired, preserved or improved?

Human Health

Consider each of the items 7.1-7.9 in relation to the local population, the labour force during construction and their camp followers, the resettled and newly settled populations and migratorylabour groups.

Water and Sanitation. Are the provisions for domestic water, sanitation and refuse disposal such that oral, faecal, water washed and other diseases and the pollution of domestic water can be controlled?

Habitation. Are the provisions for housing and forecast population densities such that diseases related to habitation or location of dwellings can be controlled?

Health services. Are general health provisions adequate (treatment, vaccination, health education, family planning and other health facilities)?

Nutrition. Is the Project leading to an increase or decrease in the general nutritional status of the population or to changes in other lifestyle or income related disease? If so, are any specific groups particularly exposed to such health risks?

Relocation effect. Are population movements introducing new infectious or water- related diseases to the Project area or causing stress-related health problems or bringing people with a low resistance to particular diseases into areas of high transmission?

Disease ecology. Are the extent and seasonal character of reservoirs, canals, drains, fast flowing water, paddy fields, flooded areas or swamps and the closeness or contact of the population with such water bodies leading to significant changes in the transmission of water related diseases?

Disease hosts. Are the populations of intermediate and other primary hosts of parasitic and water-related diseases (rodents, birds, monkeys, fish, domestic animals) and the interaction of the human population with these hosts, decreased or increased by the Project?

Disease control. Can the transmission of the diseases identified be reduced by introducing into the Project environmental modifications or manipulations or by any other sustainable control methods? Possible environmental measures include both removal of breeding, resting and hiding places of vectors and reducing contamination by and contact with humans.

Other hazards. Is the risk to the population decreased or increased with respect to:
pathogens or toxic chemicals present in irrigation water (particularly through wastewater reuse) or in the soils, which can accumulate in food crops or directly threaten the health of the population ; dwellings adequately located and designed to withstand any storm, earthquake or flood hazards; sudden surges in river flow caused by the operation of spillways or power turbines;and structures and water bodies designed to minimise accident and allow escape?

Ecological Imbalances

Pests and weeds. Are crop pests or weeds likely to increase or decrease (particularly those favoured by irrigation/drainage/flood control) affecting yields, cultivation and requirements for pesticides or herbicides?

Animal diseases. Are domestic animals in the Project or vicinity more or less exposed to hazards, diseases and parasites as a fault of the Project and its dams?

Aquatic weeds. Are reservoirs, rivers or irrigation and drainage canals likely to support aquatic vegetation or algae? If so, can these plants be harvested or controlled, or will they reduce the storage/conveyance capacity, interfere with the operation of hydraulic structures or lead to oxygen-oversaturated or anaerobic water bodies?

Structural damage. Is there a danger of significant damage being caused to dams, embankments, canal banks or other components of the irrigation/drainage/flood control works through the action of plants and animals (including rodents and termites) favoured by the Project?

Animal imbalances. Does the Project cause zoological imbalances (insects, rodents, birds and other wild animals) through habitat modification, additional food supply and shelter, extermination of predators, reduced competition or increased diseases?

जलस्रोत सम्बन्धि (१)

१२०४९जलस्रोत ऐन १.२०४९।९।१७

प्रस्तावनाः

- भूसतह भूमिगत वा अन्य कुनै अवस्थामा रहेको जलस्रोत
- समूचित उपयोग संरक्षण व्यवस्थापन र विकास
- जलस्रोतको लाभदायक उपयोगहरुको निर्धारण
- वातावरणीय र अन्य हानिकारक प्रभावको रोकथाम
- जलस्रोतलाई प्रदुषण मुक्त
-

१.संक्षिप्त नाम र प्रारम्भ २०५०।५।०१

२ ,सस्था उपभोक्ता ,अनुमतिपत्र प्राप्त व्यक्ति ,लाभदायक उपयोग ,जलश्रोत : परिभाषा . तोकिएको

३. जलस्रोतको स्वामित्वः नेपाल

४. जलस्रोतको उपयोग

अनुमतिपत्र तपसील वाहेकमा

- व्यक्तिगत वा सामुहिक खानेपानी वा घरेलु प्रयोजन
- व्यक्तिगत वा सामूहिक सिंचाइ
- घरेलु उद्योगलाई पानीघट्ट वा पानी चक्की
- स्थानीय आवागमनलाई व्यक्तिगत डुंगा
- आफ्नो जग्गामा घरधनीले

५. जल उपभोक्ता संस्थाको गठन

- संस्था गठन गर्न सक्ने

६. उपभोक्ता संस्था संगठित संस्था हुने

- अविच्छिन्न उत्तराधिकारवाला स्वशासित र संगठित
- छुट्टै छाप
- व्यक्ति सरह चल, अचल सम्पति प्राप्त, उपभोग, वेचविखन
- व्यक्ति सरह नालिस उजुर गर्न र लाग्न

७. जलस्रोत उपयोगको प्राथमिकता

- खानेपानी र घरेलु उपयोग
- सिंचाइ

- पशुपालन तथा मत्स्यपालन जस्ता कृषिजन्य उपयोग
- जलविधुत
- घरेलु उद्योग औधोगिक व्यवस्था वा खानीजन्य उपयोग
- जल यातायात
- आमोद प्रमोद जन्य उपयोग
- अन्य उपयोग

८. **अनुमतीपत्रको व्यवस्थाः**
- सर्वेक्षण वा उपयोग गर्न चाहने
- आर्थिक, प्राविधक र वातावरणीय अध्ययन प्रतिवेदन(उपयोगको अनुमतिको लागि आवश्यक)
- जलस्रोत सेर्भेक्षणको अनुमति- ३० दिन, उपयोगको- १२० दिन (अनुमति दिनु पर्ने)
- ऐन जारी हुनु भन्दा उपयोग गरेको हकमा पएक वर्ष भित्र निवेदन दिने
- ६० दिन भित्र अनुमति
- दस्तुर लाग्ने
- हस्तान्तरणको लागि वा विक्रीको स्वीकृति

९. **जलविधुतको लागि जलस्रोतको उपयोगः**
- प्रचलित कानून वमोजिम

१०. **नेपाल सरकारले जलस्रोतको उपयोग वा विकास गर्न सक्नेः**
- नेपाल सरकारले आफै गर्न सक्ने
- जग्गा ,भवन, उपकरण तथा संरचना आफै विकास गर्ने
- प्राप्तको लागि क्षतिपूर्ति दिने
- वर्तमान मूल्यको आधारमा क्षतिपूर्ति

१०क. **प्रदेश र स्थानीय तहले जलस्रोतको उपयोग वा विकास गर्न सक्नेः**
- राजपत्र सूचना प्रकाशन गरी तोकि दिए वमोजिमका स्तर क्षमता वा परिमाणका खानेपानी जलविधुत सिंचाइ लगायत जलस्रोतको सम्बन्धमा प्रदेश वा स्थानीय तहले कानून वनाइ त्यस्तो कानून वमोजिम जलस्रोतको उपयोग वा विकास गर्न सक्नेछ ।

११. **जलस्रोतको विकास परियोजना हस्तान्तरण गर्न सक्नेः**
- उपभोक्ता समितिलाई हस्तान्तरण
- स्वामित्व उपभोक्ता समितिलाई हस्तान्तरण

१२. **जलस्रोतको उपयोगको लागि करार गर्न सक्नेः**

- कुनै स्वदेशी वा विदेशी कम्पनी संगठित संस्थगा वा व्यक्ति सित प्रचलित कानूनको अधिनमा रही करार गरी जलस्रोतको विकास, उपयोग र सेवा विस्तार

१३ सेवा उपयोगका शर्तहरु तोक्न र सेवा शुल्क असुलउपर गर्न पाउनेः

- जलस्रोतको उपयोगको सेवा, शर्त र शुल्क असुलको आधारमा

१४. सेवा रोक्न सकिनेः

- शुल्क नवुझाउने, अनिधिकृत प्रयोग दुरुपयोग, शर्त विपरीत सेवा उपयोगगरेमा

१५. अरुको घरजग्गामा प्रवेश गर्न सक्नेः

- पूर्व सूचना दिएर प्रवेश गर्न पाउने (सर्भेक्षण वा उपयोग गर्ने सिलसिलामा)
- हानीनोक्सानी भए सरकारले क्षतिपूर्ति दिने
- दुरुपयोग वा अनाधिकृत उपयोगमा पूर्व सूचना आवश्कय नहुने

१६. अरुको घरजग्गाको उपयोग वा प्राप्तीः

- प्राप्त गर्न सकिने अवस्थाः (निवेदन दिने)
  - क) बाँध वा तटवन्ध वाँध्न
  - ख) नहर, कुलो वा सुरुङ्ग खन्न
  - ग) जमीन माथि वा मुनी पानीको टयाङ्की वनाउन वा पाईप लाईन विछ्याउन
  - घ) पोखरी वनाउन वा जल वितरण केन्द्र स्थापना गर्न
  - ङ) जलस्रोतको विकास सित सम्बन्धित अरु आवश्यक निर्माण
- नेपाल सरकारले जग्गा प्राप्त गरी अनुमति प्राप्त संस्थालाई दिने
- अरुको जग्गा प्रयोग गर्न नपाउने गरी निषेध गर्ने
- त्यस वापत क्षतिपूर्ति दिने

१७. जलस्रोतको उपयोगसित सम्वन्धित संरचनाको सूरक्षाः

- नेपाल सरकारले सूरक्षा दिने अनुमतिपत्र प्राप्त व्यक्तिको अनुरोध र खर्चमा

१८. जलस्रोतको गुणस्तर तोक्नेः

- नेपाल सरकारले राजपत्र मार्फत तोक्ने

१९. जलस्रोतलाई प्रदुषित गर्न नहुनेः

- जलस्रोतको प्रदुषण सहन सिमा तोक्ने
- प्रदुषण सिमा नाघ्ने गरी कसैले पनि पकुनै किसिमको फोहोर मैला औद्योगिक निकास विष रासायनिक वा विषालु पदार्थ हाली वा प्रयोग गरी जलस्रोतलाई प्रदुषित गर्नु हुदैन
- गुणस्तर भए नभएको परीक्षण गर्न सकिनेछ

२०. वातारवरणमा उल्लेखनीय प्रतिकुल असर पार्न नहुनेः

- जलस्रोतको उपयोग गर्दा भूक्षय, वाढी, पहिरो वा यस्तै अरु कारणवाट वातावरणमा उल्लेखनीय प्रतिकूल असर नपर्ने गरी गर्नु पर्नेछ ।

२१. **अनुमतिपत्र खारेज गर्न सकिनेः**

- नियम विपरीत काम भएमा सुधार गर्न आदेश
- सुधार नभएमा अनुमतिपत्र खारेज
- सफाई पेश गर्ने मौका

२२. **दण्ड सजायः**

- नियम उलंघन भएमा ५ हजार जरिवाना र भएको हानि नोक्सानी
- अनुमतिपत्रको शर्त पालना नभएमा पाँच हजार र काम वन्द
- चोरी, दुरुपयोग वा अनाधिकार प्रयोग- **विगो+विगो वरावरको जरिवाना**
- संरचनामा क्षति विगो+विगो वरावर जरिवाना वा दश वर्ष कैद वा दुवै

२३. **पुनरावेदनः**

- दफा १०,१५, वा १६ को क्षतिपूर्ति दफा २१ वमोजिमको खारेजी, दफा २२ का १,२,३ वमोजिमको सजाय चित्त नवुझेमा उच्च अदालतमा पुनरावेदन दिन सकिने

२४. **नियम वनाउने अधिकारः**

- उद्देश्यलाई प्रतिकूल असर नपर्ने गरी
    - जलस्रोत को उपयोग सम्बन्धि
    - जलस्रोत संरक्षण बाढी नियन्त्रण र पहिरो रोकथाम सम्बन्धि ,
    - बाताबरण सम्बन्धि
    - सेवा वापत बुझाउनु पर्ने सुल्क सम्बन्धि
    - प्रदूषण रोक्ने सम्बन्धि
    - उपयोग को तरिका सम्बन्धि
    - स्तर निर्धारण सम्बन्धि
    - दुर्घटना ,जाचबुझ र क्षतिपुर्ती सम्बन्धि
    - उपभोक्ता सस्था संरक्षण र सुबिधा सम्बन्धि

२५ खारेजी र बचाउ .

- नहर तथा बिधुत र तत् सम्बन्धि ऐन २०२४ खारेज

१जलस्रोत नियमावली २., २०५०

परिच्छेद

१. प्रारम्भिक

२. उपभोक्ता संस्था

३. जलस्रोत उपयोग सम्बन्धि व्यवस्था

४. जलस्रोत उपयोग विवाद स

५. सेवा शुल्क

६. घर जग्गा प्राप्ती वा क्षतिपूर्ति

७. विविध

परिच्छेद १ प्रारम्भिक

१. संक्षिप्त नाम र प्रारम्भ- जलस्रवत नियमावली, २०५०

२. परिभाषाः

परिच्छेद-२ उपभोक्ता संस्था

३. उपभोक्ता संस्थाको गठनः

- कम्तिमा सात सदस्य भएको संस्था गठन गर्न सकिनेछ

४. उपभोक्ता संस्था दर्ताको लागि दरखास्त दिनुपर्ने

- कम्तीमा सात सदस्य एक प्रति विधान र रु १०० (सय) रुपैया दस्तुर समेत जिल्ला जलस्रोत समितिमा दरखास्त दिनुपर्ने

५. विधानमा खुलाउनु पर्ने विवरणहरुः (७३)

- पुरा पनाम र ठेगानाः
- उद्देश्य र कार्यक्षेत्र
- सदस्यको लागि योग्यता र सदस्यता शुल्क
- सदस्यको निष्काशन र राजिनामा
- हक दावी नामसारी वा हकवालाको मनोनयन
- साधारण सभा सम्वन्धि
- संचालक समितिको गठन
- संचालक पदमा वहाल रहन नसक्ने अवस्था
- संचालक समितिको वैठक सम्वन्धि कार्यविधि
- कोष र लेखा परीक्षण
- विधान संशोधन
- विघटन
- विविध

६. दर्ता प्रमाणपत्रः

- जिससले प्रमाण दिने
- दर्ता पहुन नसकेमा तिस दिन भित्र सूचना
- यो नियमावली जारी हुनु भन्दा अगाडी दर्ता भएकालाई यसै वमोजिम भएको मान्यता

७. विधान संशोधनः

- जिससमा पेश गर्ने र स्वीकृति लिने

परिच्छेद ३ जलस्रोत उपयोग सम्बन्धि व्यवस्थाः

८. जिल्ला जलस्रोत समितिको गठनः

- प्रमुख जिल्ला अधिकारी– अध्यक्ष
- प्रतिनिधि जिकृविका– सदस्य
- प्रतिनिधि जिल्ला वन कार्यालय– सदस्य
- प्रतिनिधि जिल्ला खानेपानी कार्यालय– सदस्य
- प्रतिनिधि जिल्ला सिंचाइ कार्यालय– सदस्य
- प्रतिनिधि सम्बन्धित जिल्ला भित्रका नेपाल सरकारद्वारा संचालित विधुत आयोजनाको कार्यालय – सदस्य
- जलस्रोत उपयोग गर्ने अन्य कार्यालयको प्रतिनिधि– सदस्य
- प्रतिनिधि जिल्ला विकास अधिकारी– सदस्य सचिव

९. जिल्ला जलस्रोत समितिको वैठक सम्बन्धि कार्यविधिः

- अध्यक्षले तोकेको समय स्थान र मिति
- अध्यक्ष नभएमा सहमतिमा छानिएको व्यक्तिले अध्यक्षता
- पचास प्रतिशत गणपूरक संख्या
- मतदान हुँदा उपस्थितको दुई तिहाईवाट निर्णय
- निर्णय सदस्य सचिवले प्रमाणित गर्ने
- अन्य कार्यविधि आफै तोक्ने

१०. जिल्ला जलस्रोत समितिको संयुक्त वैठक

- एक भन्दा वढी जिल्ला संग सम्वन्धित भए संयुक्त वैठक
- प्रमुख संरचना रहने जिल्लाले अध्यक्षता गर्ने भाग

११. जिल्ला जलस्रोत समितिको सचिवालय

- स्थानीय विकास अधिकारीको कार्यालय

१२. जलस्रोतको

उपभोगको सर्भेक्षण अनुमतिपत्रको लागि दस्तखत दिनेः

- जिल्ला जलस्रोत समितिमा दिने
- आवश्कय कागजात–

परियोजना

को विवरण परियोजना रहने स्थानको नक्सा पानीको स्रोत र उपभोग हुने पानीको परिमाण लाभान्वित हुने उपभोक्ताहरुको संख्या र किसिम सर्भेक्षण गरिने जलस्रोतको क्षेत्र कुल लागत अवधि र अन्य

- दस्तुर लाग्ने

१३. दरखपास्त

उपर जाँचवुझः

- जाँच गर्ने कागजात नपुगेमा १५ दिन भित्र सूचना दिने र पुनः पेश गरेको मितिलाई दरखास्त मिति मान्ने

१४. सर्भेक्षण

अनुमतिपत्र दिने

- जिल्ला जलस्रोत समितिले छानवीन पछि दिने

१५. सर्भेक्षण

प्रतिवेदन पेश गर्नुपर्ने

- सम्पन्न भएको तिस दिन भित्र तिन प्रति

१६. दोहोरो

सर्वेक्षण अनुमतिपत्र नदिइने

१७. जलस्रोत

उपभोगको अनुमतिपत्रको लागि दरखास्त दिने

- जिल्ला जलस्रोत समितिमा तिन प्रति दरखास्त दिने
- आवश्यक विवरण परियोजनाको विस्तृत विवरण सम्भाव्यताको विष्लेशन वितिइ व्यवस्था घर जग्गाको उपभोग वा प्राप्ती वातावरणीय प्रभाव विश्लेषण र अन्य आवश्कय कुराहरु

१८. दरखास्त

उपर जाँचवुझः

  - जाँचवुझ गर्ने थप विवरण तिस दिन भित्र माग गर्ने, पुनः वितरण प्राप्त भएको मितिलाई दरखास्त मिति मान्ने

१९. सार्वजनिक

सूचना प्रकाशित गर्नुपर्ने

- १७ र १८ को प्रकृया पुरा पभएपछि सार्वजनिक जानकारीको लागि सूचना प्रकाशित गर्नुपर्ने
- पैतिस दिन भित्र जो सुकैले पनि मन्तव्य दिन मिल्ने
- प्राप्त प्रतिक्रियालाई विचार गरेर अनुमतिपत्र दिंदा तोक्ने

२०. अनुमिति पत्र दिने

- १७ वमोजिमको निवेदन १८ र १९ को कार्य विधि पुर्याएर अनुमति पत्र दिने

२१. २१।

सर्वेक्षणको अनुमित प्राप्त व्यक्तिलाई परियोजन संचालन गर्न अनुमति पत्र प्रदान गरिने

- प्राथमिकता दिने

२२. जलस्रोत माथि अधिकार कायम हुने ।

- तोकिएको स्थान र क्षेत्रसम्मको जलस्रोतको उपयोग

२३. कार्य पशुरु गर्नुपर्ने अवधि

- सर्वेक्षणको हकमा तीन महशिना उपभोगको हकमा एक वर्ष
- नसकेमा उचित कारण सहित म्याद थप ग९राउन सकिने
- कामको प्रगति प्रत्येक ६ महिनामा प्रगति पेश गर्ने

२४. नयाँ अनुमतिपत्र लिनु पर्ने

- ऐन प्राम्भ भए अगाडी उपभोग गरि रहेको अवस्थामा पुनः पअनुमतिपत्र लिनुपर्ने

२५. अनुमतिपत्र दस्तुरः

- अनुसूचि अनुसार

२६. अनुमतिपत्र नवीकरण गर्नेः

- अनुमतिपत्रमा उर्ल्लेखित समय समाप्त हुनु अगावै अर्को अवधिको लागि नवीकरण

२७. अनुमतिपत्र विक्री वा हस्तान्तरण गर्न स्वीकृति लिनु पर्ने

- विक्री वा हस्तान्तरण गर्नुपरेमा जिल्लाप जलस्रोत समितिमा निवेदन दिनुपर्ने
- जाँचवुझ गरेर विक्री वा हस्तान्तरणको अनुमति

परिच्छेद ४– जलस्रोत उपभोगको विवाद सम्बन्धि जाँचवुझः

२८. जलस्रोत

उपयोग जाँचवुझ समितिः

- समितिः- प्रतिनिधि,

  जलस्रोत मन्त्रालय- अध्यक्ष

  प्रतिनिधि, सम्वन्धित जि.वि.स.- सदस्य

  प्रतिनिधि, रायोआ को क्षेत्रीय कार्यालय- सदस्य

- दुइ वा दुइ भन्दा वढी जिल्लाको सम्बन्धित विवाद उत्पन्न भएमा प्रत्येक जिल्लाका जिल्ला विकास समितिका प्रतिनिधि
- जाँचवूझ समितिको कार्यविधि
- प्राथमिकता निर्धारण वा परियोजना संचालन सम्वन्धमा विवाद भएमा अनुमतिपत्र प्राप्त व्यक्तिसंग म्याद तोकी लिखित जानकारी पेश गर्न लगाउने
- जानकारी लिनुपर्ने परियोजनाको कुल खर्च फाइदाको स्तर व्यक्ति वा समूहलाई पर्न सक्ने असर लाभान्वित उपभोक्ताको संख्या वातावरणमा पर्ने प्रभाव स्थानी जनताको आवश्यकता लाभान्वित हुने उपभोक्ताहरुको प्रतिक्रिया वा अन्य आवश्यक कुराहरु
- उल्लेखित विषयहरुको जानकारी प्राप्त गरी निर्णय गर्ने
- शर्तहरु तोक्न सक्ने

परिच्छेद ५- सेवा शुल्क सम्बन्धि

२९. बार्षिक

शुल्कः व्यापारिक प्रयोजनको लागि सेवा उपलब्ध गराए वापत वार्षिक शुल्क तिर्नु पर्ने

३०. सेवा

पशुल्क निर्धारण समितः

- नेपाल सरकारले तोकेको व्यक्ति
- उपभोक्ताहरु मध्ये नेपाल सरकारले तोकेको व्यक्ति- सदस्य
- नेपाल सरकारले तोकेको व्यक्ति- सदस्य

  सचिव

- कार्यविधि आफैले तोकेने
- शुल्क निर्धारणका आधारहरु - ह्रासकट्टी पदर उपयुक्त लाभ संरचनाको तरिका उपभोक्ता मुल्य सूचीको परिवर्तन

३१. सेवा शुल्क

बुझाउनु पर्ने

परिच्छेद ६ घर जग्गा प्राप्ति तथा क्षतिपूर्ति सम्बन्धि व्यवस्था

३२. निवेदन

दिनु पर्ने

- निवेदन दिनु पर्ने

३३. परियोजना

स्थल विरपरीको जग्गा प्रयोग गर्न निषेध गर्न सक्ने

- निषेध गर्ने सक्ने
- सुचना टाँस गर्ने

३४. क्षतिपूर्ति

दिइने

- घर जग्गा प्रयोग गर्न निषेध गरिएको कारणवाट सम्बन्धित व्यक्तिलाई भएको हानी नोक्सानी वापत दिइने रकम समितिले निर्धारण गर वमोजिम हुनेछ ।

३५. क्षतिपूर्ति

निर्धारण समितिः

- नेपाल सरकारले तोकेको व्यक्ति- अध्यक्ष

जलस्रोत उपयोग सम्बन्धित परियोजना कार्यालय प्रतिनिधि- सदस्य

जलस्रोत सम्बन्धि नेपाल सरकारले तोकेको विशेषज्ञ- सदस्य

ऐनको दफा १५ र १६ को लागि थप सदस्यहरु हानी नोक्सानी भएको अचल सम्पतिको धनिक वा निजको प्रतिनिधि- सदस्य

सम्बन्धित जिल्ला मालपोत कार्यालयको प्रतिनिधि- सदस्य

वास्तविक हानी नोक्सानीको आधारमा क्षतिपूर्तपि

परिच्छेद ७ विविध

३६. अधिकारी

तोकिएको

- ऐनको दफा २१ जिल्ला जलस्रोत समिति- अनुमति

पत्र खारेज

- दफा १८ (१) राजपत्रमा तोके अनुसार- गुणस्तर

ऐनको दफा २२ (१) (२) (३) जलस्रोत सचिव- सचिव दण्ड

सजाय

३७. सहयोग

पुर्याउनु पर्ने

- जिल्ला जलस्रोत समितिले माग गरेका तथ्याङ्क उपलब्ध गराउन

३८. दुर्घटना

सूचनाः

तुरुन्त जिल्ला जलस्रोत समितिलाई सूचना दिने कारण पत्रा लगाउन निरीक्षण गर्ने। गराउने पदुर्घटना दोहोरिन नदिन सुरक्षात्मक व्यवस्थाको आद्श दिन सक्ने अनुमति प्राप्त व्यक्ति आदेश मान्नु पर्ने

३९. नेपाल

सरकारले निर्देशन दिन सक्ने

- जिल्ला जलस्रोत समितिलाई

४०. अनुसूची

फेरवदल थपघट गर्नप सक्ने

अनुसूची- ८ अनुमतिपत्र दस्तुर

खानेपानी र घरेलु उपयोग	१००
सिंचाइ	२००
पशुपालन तथा मत्स्यपालन	१००
घरेलु उपयोग	२००
जल यातायात	५००
आमोद प्रमोद	५००
अन्य उपयोग	५००

अनुसूचि ९ उपभोषग वापतको दस्तुर

१. सिंचाइको हकमा

<१००० हेक्टर- Rs 2/ha - max.
1000

1000- 5000 -	2000
5000 - 10000	5000
10000 - 15000	10000
15000 -	20000 15000
20000- 25000	20000
>25000	25000

२. खानेपानीको हकमा

< 2000 population -	500
2000 - 5000	2000
5000 - 10000	5000
10000 - 15000	10000

15000 - 20000 15000
20000 - 25000 20000
>25000

25000

३. कृषिजन्य उपभोग-

000-

२0000

४. औधोगिक व्यवसाय तथा खानी नज्य उपयोग- ५000-

४0000

५. आमोद प्रमोदजन्य-

१000-

१0000

६. अन्य

५000-

१0000

१.३ Water Resources Strategy, Nepal 2002

1. **Purpose of the Water Resources strategy**
   - Every Nepali should have access to water sufficient to meet basic needs.
   - Sufficient water available.
   - Progress is slow.
   - Increasing water use complies
   - WRS involves the reconciliation of a range of problems and constraints to sustainable water resource development, including those related to government policies, financial and human resources, institutions and actions.

2. **Strategy formulations process.**
   - Participatory log frame approach.
   - Development of overall strategic goal, identification of short,medium and long term purpose to contribute that goal and the definition of ten strategic output.
   - Short term & tangible benefits to all Nepalese by improving their access to water sufficient to meet basic needs. (5 Yrs.)
   - Midterm(15 Yrs.) – Sustainable water use and other aspects of strategy are realized.
   - Long Term – Maximize the benefit from water resource ultimately improve Nepal living condition.

Pre-conditions:
   - The water resource sector & water resource strategy will receive continued high priority & support from Government.
   - All the stakeholders including political parties will support the strategy & its implementation.

3. **Water sector needs & issues:**
   - 66% to safe water.
   - 41% of irrigated land year-round irrigation.
   - <400 MW hydropower available.
   - Little consideration to environmental requirements.

Irrigation Issues:
   - Reorientation of supply driven approach.
   - Poor performance of irrigation systems
   - Lack of effective implementation of APP.
   - Farmers dependency syndromes& sustainability.
   - Problems of river management.
   - Well institutional capability.
   - Symbolic relationship between agriculture &Irrigation.
   - Strengthening of Water User Associations (WUAs)

4. **Policy framework adopted for WRS formulation**
   Specific objectives adopted for WRSF include.
   - Help reduce the incidence of poverty, unemployment and under employment.
   - Provide access to safe and adequate drinking water and sanitation for ensuring health security.
   - Increase agricultural production, ensuring the nation’s food security.

- Generate hydro power to satisfy national energy requirements and to allow for export of surplus energy.
- Supply the needs of the industrial sector and other sectors of the economy.
- Facilitate water transport, particularly connection to a seaport.
- Protect the environment & sustain the biodiversity of natural habitat.
- Prevent & mitigate water induced disasters.

Policy Principles:

- Integrated water resource management.
- Sustainable
- Decentralized
- Economic efficiency & social equity.
- Participation and consultation of all stakeholders.
- Sharing of water resources.
- Institutional and legal frameworks.
- Wider adoption

Guiding Principles:

- Social development principles.
- Economic development principle.
- Environmental sustainability principles.

5. Water Resource Strategy:

National Goal – Living conditions of Nepali people are significantly improved in a sustainable manner.
Short term (5 Yrs.) – Implementation of comprehensive WRS provides tangible benefits to the people in live with basic needs fulfilments, supported and managed by capable institutions involving all stakeholders.
Medium Term (15 Yrs.) – The WRS is operationalized to provide subs trial benefits to people for basic needs fulfillment as well as other increased benefits related to sustainable water use.
Long Term (25 Yrs.) – Benefits from water resources are maximized in Nepal in a sustainable manner.

Categories:

1. Security (Outputs 1 and 2) – Security from water induced impacts and security of water supply,
2. Uses (Outputs 3 to 6) – types of water use.
3. Mechanisms (Outputs 7 to 10) – mechanism

Outputs

1. Effective measures to manage and mitigate water induced disasters are functional.
2. Sustainable management of waters needs and Aquatic ecosystems achieved
3. Adequate supply of and access to potable water, sanitation and hygiene awareness provided.
4. Appropriate and efficient irrigation available to support optimal.
5. Cost Effective Hydropower developed in Sustained Manner
6. Economic use of water by industries and water bodies by Tourism, Fisheries & Navigation optimized.
7. Regional co-operation for sustainable mutual benefits achieved.
8. Enhanced water related information systems are function.

9. Appropriate legal frameworks are functional.
10. Appropriate institutional mechanisms for water sector management are functional.

Output 1: Effective measures to manage and mitigate water induced disasters are functional.

Activities:

- ➢ Prepare and implement a water induced disaster management policy and plan.
- ➢ Conduct risk/vulnerability mapping and zoning.
- ➢ Strengthen the disaster networking and information system establish disaster relief and rehabilitation system.
- ➢ Carry out community awareness/education on disaster management.
- ➢ Activate Inundation committee w.r.t.neighboring countries.
- ➢ Prepare and implement flood plan action plans
- ➢ Implement disaster reduction/mitigation measures.
- ➢ Strengthen institutional set up and capacity.

Indicators:

- ➢ 2007 – potential disaster zones identified by type and located on district maps.
  - ▪ Emergency relief materials are available in all five regions.
- ➢ 2017 – infrastructure for mitigating predictable disasters put in place in 20 districts.
  - ▪ Warning systems established and functioning, encompassing the country.
- ➢ 2027 – Social and economic losses reduced to levels experienced in developed countries.

Output – 2: Sustainable management of waters needs and Aquatic ecosystems achieved.

Activities:

- ➢ Improve environmental data base system.
- ➢ Map important, critical and priority waterheads and aquatic ecosystems.
- ➢ Develop water & waste water quality standards and regulation.
- ➢ Implement a water conservation education programme.
- ➢ Utilize strategic environmental assessment in water resources management.
- ➢ Ensure compliance with environmental regulations.
- ➢ Ensure community participation.
- ➢ Increase potential capacity and co-ordination.

Indicators:

2007 – management plan for pilot watershed and aquatic system prepared and initiated.

- ➢ Water quality and waste water quality standards developed and initiated.

2017 – Full scale environmental protection and management projects implemented in all priority watersheds and aquatic ecosystems.

- ➢ Stakeholders participating in environmental protection and management.

2027 – Quality of watershed increased by 80% in all regions.

- ➢ Adequate water quality for aquatic habitat including fish, human consumption and recreation in all rivers and lakes.

Output – 3: Adequate supply of and access to potable water, sanitation and hygiene awareness provided.

Output – 4: Appropriate and efficient irrigation available to support optimal.

Activities:

- Integrate irrigation planning & management water agricultural development.
- Improve management of existing irrigation systems.
- Develop year sound irrigation in support of intensification and diversification of agriculture.
- Strengthen local capacity for planning, implementations and management of irrigation.
- Encourage consolidation of land to promote irrigation/agricultural efficiency.
- Improve grou9nd water development & management.

Indicators:

2007 – Year-round irrigation increased to 50% of irrigated land.

- All AMIS managed jointly with WUA

2017 – year-round irrigation increased to 66% of irrigated land.

- 80% of all irrigable land sieved by irrigation schemes.
- APP target regarding irrigation achieved.

2027 – 90% of all irrigable land provided with year-round irrigation.

- Irrigation efficiency increased to 60%.
- Nepal's food security maintained throughout the 25-year strategy period.

Output 5: Cost Effective Hydropower Developed in Sustainable Manner

Output – 6: Economic use of water by industries and water bodies by Tourism, Fisheries & Navigation optimized.

Output – 7: Regional co-operation for sustainable mutual benefits achieved.

Output – 8: Enhanced water related information systems are function.

Output – 9: Appropriate legal frameworks are functional.

output – 10: Appropriate institutional mechanisms for water sector management are functional.

6. Resources required for strategy implementation recommended institutional changes

DOI – Re-Oriente& transfer of flood control mandate to other departments.	- Continue to reorient staff to facilitate community participation & ownership of schemes.
	- Focus on multipurpose project planning & implementation.
	- Focus on conjunctive use of surface & GW to achieve year round irrigation starts.

GW regulation Authority:

- Convert the existing GW board to a regulating authority to monitor, regulate & investigate GW potential.	- Increase capability of staff to license &monitor GW use, including water quality.
-	- Increase capability of staff to investigate & maintain a scientific database of GW aquifers.

DSCWM:

- No change but designate as lead	- Strengthen mandate to control and

agency to protect & enhance watersheds.	protect watersheds.
-	- Increased funding of programs to enhance watersheds.

DWIDP:

Expanded mandate as lead agency for all water related disasters	- Strengthen mandate to co-ordinate disaster prevention & mitigation measures.
	- Transfer flood control responsibility & staff from DOI.
	- Increased finding for development programs.

Financial Commitments Required:

- The budget for the irrigation sub-sector will remain at recent levels but needs to be targeted at projects that provide better economic returns.
- Increased finding is required for water induced disaster prevention.

Cost Recovery for Irrigation Sub-Sector:

- Already moved for ground water irrigation
- WUA established for recovery of operational lost but not he capital cost.

Total Investment Required:

Irrigation – 145.9 billion.

Disaster prevention – 56 billion.

१.४ National Water Plan 2005

Part A: National Water Plan: The context.

1. General Context.

Physical Setting:

- Five physiographic regions – high Himalayas, lesser Himalayas (Mountains), Middle Mountain (the Mahabharat Range), Siwaliks (The Churia Range) and the Terai Plains.
- Climate – Sub-tropical to alpine.
- Elevation – 64m. – 8848 m
- Snowfalls in mountain.
- Average precipitation – 1530 mm

Social Setting:

- > Sixty caste & ethnical group – side – Aryan and Tibet's – Mongolidstocus.
- HD1~ 0.504
- GDP 248$ - 2005
- Population projection for 2026-27 – 37.7 millions.

Economic Setting:

- Agriculture based economy

Country's Water Resources.

- 6000 rivers.
- Four major rivers – Mahakali, Karnali, Narayani&Koshi.
- 5 medium rivers – Kankai, Kamala, Bagmati, West Rapti&Babai
- Total average runoff ~ 225 billion cubic meter.
- Ground water recharge ~ 600 mm/yr, 400 mm/yr recoverable
- Rechargeableground waters ~ 8.4 – 11.5 13cm
- Present – 756 million cubic meter – irrigation
  297 million cubic meter – domestic use.

Existing Water Use Scenario:

- Only 15 BCM out of 225 BCM utilized for economic and social purposes.
- 72% population access to basic water supply.
  25% population sanitation facility.
- Irrigation – 2.64 million ha cultivable land
  million ha irrigable
  1.76 Million ha irrigable
  60% with irrigation facility
- 70% area FMIS
- Electricity – 40% of population

2. Planning Context:

Planning efforts in the past.

- Nepal in planned process of development in 1956.
- Sectoral plan APP@ Foresting master plan, Irrigation master plan, River basin master plan, Tourism Master Plan and other sectoral plan.

Rational of Water Plan:

- Water as principle natural source.

- Water resource development is slow and unable to contribute much towards the alleviation of poverty.
- Water resource strategy developed in 2002
- NWP in live with WRS

Need for IWRM and RBM:

- Traditional water resource management.
- Development and management of water resources shall be undertaken in a holistic & systematic manner.
- Water utilization shall be sustainable to ensure conservation of resources & protection of the environment. Each river basin system shall be managed holistically.

River Basin Management for

- The enabling environment
- Institutional framework.
- Management Instrument.

Consultation Process Adopted or NWP:

- PRTICIPATION AND CONSULTATION

Water Sector objectives and Policy framework:

Water Sector Objectives:

- To help reduce the incidence of poverty, unemployment and under employment.
- To provide people with access to safe and adequate drinking water and sanitation for ensuring health security.
- To increase agricultural production and productivity, ensuring food security of the nation
- To generate hydropower to satisfy national energy requirement and to allow export of the industrial and other sector of the economy.
- To supply the needs of the industrial and other sector of the economy.
- To facilitate water transport, particularly connection to a sea port.
- To protect the environment and conserve the biodiversity of natural habitat.
- To prevent and mitigate water induced disaster

Policy Principles.

- IWRM
- Sustainable
- Decentralized
- Economic efficiency and social equity.
- Participation
- Sharing
- Institutional & Legal.
- Wider adoption

Other Guiding Principles of Strategy Formulation:

- Social Development
- Economic development
- Environmental sustainability

The Water Strategy: Salient Features:

Short term (5 yrs.) purpose: - Provides tangible benefits for people in line with basic needs fulfillment.

Medium Term (15 Yrs) Purpose: - to provide substantial benefits for the basic needs fulfilment of the people as well as other increased benefits related to sustainable water use.

Long Term (25 Years) Purpose: - benefits from water resources are maximized in Nepal in a sustainable manner.

Strategic Outputs:

Security:

1. Effective measures to manage and mitigate water induced disaster are made functional.
2. Sustainable management of water shades and aquatic ecosystem is achieved.

Use:

3. Adequate supply of and access to potable water and sanitation and hygiene awareness is provided.
4. Appropriate and efficient irrigation is made available to support optimal and sustainable use of irrigable land.
5. Cost effective hydropower is developed in a sustainable manner.
6. Economic use of water by industries and water bodies by8 tourism, fisheries and navigation are optimized.

Mechanisms:

7. Enhanced water related information systems are made functional.
8. Appropriate legal frameworks are made functional.
9. Regional co-operation for substantial mutual benefits is achieved.
10. Appropriate institutional mechanisms for water sector management are made functional.

Part B: The National Water Plan

4. The NWP

- Contribute in a balance manner to the overall national goals of economic development, poverty alleviation, food security public health and safety, decent standards of living for the people and protection of the natural environment.

4.1 Doctrines of NWP

- Integration
- Co-ordination
- Decentralization
- Popular participation
- Equity
- Good Governance

4.2 Sub-Sectoral Programme:

Security Aspects.

- Water Induced Disasters.

Targets:

2007 – Potential disaster zones are identified by type and located on district maps.

Emergency relief materials are available in all five regions.

2017 – Infrastructures for mitigating predicable disasters are put in place in twenty districts.

- Warning systems are established and made functional, encompassing the whole country.
- Social & economic losses due to water induced disaster are reduced to the levels experienced in other developed countries.

Action Programs:

1. Water related disaster management policy & program.
2. Risk/vulnerable mapping and zoning program.
3. Disaster networking and information system improvement program.
4. Relief and rehabitation measures.
5. Community led disaster preparties program
6. Activation of inundation committee.
7. Flood, drought, slides/debris flow, GLOF and avalanche mitigation program.

Total estimated cost – 35038 (million) ➧How much budget till now??

Environmental Action Plan on Management of Watersheds and Aquatic Ecosystem:

Targets:

2007 – Water Management plan for nationally important watershed and aquatic system is prepared & initiated.

- Water quality and waste water quality standards are developed and enforced.

2017 – Full scale environmental protection and management projects are implemented in all priority watershed and aquatic ecosystems.

- Stakeholders are participating in environmental protection and management.

2027 – Quality of watersheds is increased by 80% in all regions.

- Adequate water quality aquatic habitat, including, fish, human consumption and recreation is ensured in all rivers and lakes.

Action Program:

1. Improve environmental database system.
2. Map important, critical and priority watersheds and aquatic ecosystem.
3. Develop water and wastewater quality standards and regulations.
4. Implement water conservation education program
5. Implement nationally important watersheds and aquatic ecosystem protection, rehabilitation and management program.
6. Develop strategic environmental assessment in water resources management.
7. Ensure compliance with EIA
8. Promote community participation in the management of watersheds and aquatic ecosystems.
9. Enhance institutional capacity & co-ordination
10. Develop watershed management policy.

4.2.2 Use Aspects:

- Water supply, sanitation & Hygiene
- Irrigation for agriculture

Target

- 2007 – Year round irrigation to 49% of total irrigated area
  - Average cereal yield in irrigated area increases by 15% over the 2001 level.
  - Average cropping intensity exceeds 140$ in year round irrigated areas
  - Average cropping intensity of cereal crops exceeds 126% & overall cropping intensity, including that of other crops, exceeds 160%
  - Seventy one percent of the potential area is served by irrigation systems.
  - Irrigation Efficiency increased to 35% of the O & M Cost
- 2017 –Year round irrigation is provided to 64% of the total irrigated area.
  - Average cereal yield in irrigated area increases by 28% over the 2001 level.

- Average cropping intensity exceeds 164% in year round irrigated area.
- Average cropping intensity of cereal crops exceeds 134% & over all cropping intensity, including that of other crops, exceeds 170%.
- Eighty percent of the potential area is sieved by irrigation systems.
- Irrigation efficiency increases to 45%
- ISF contribution increases by 45% of the O & M cost.

2027 – Year round irrigation is provided to 67% in the total irrigated area
- Average cereal yield in irrigated area increases for 44% over the 2001 level.
- Average cropping intensity exceeds 193% in year round irrigated areas
- Average cropping intensity of cereal crops exceeds 143% with respect to entire cultivated are and overall cropping intensity, including that of other crops, exceeds 200%
- 91% of the potential irrigable area is sieved by irrigation system.
- Irrigation efficiency increases to 50%
- Service contribution collection increases by 75% of the O & M cost.

Action Programs:
1. Integrated program for irrigated agriculture.
2. Improved management of existing irrigation scheme.
3. Improved planning & implementation of New Irrigation Systems.
4. Strengthening of capacity building of local level institutional in planning and project implementations.
5. National capacity building of farmers.
- Hydropower Development
- Industries, tourism, fisheries & navigational usese

4.2.3 Mechanisms:
- Water Related information system
- Reginal co-operation frameworks
- Legal frameworks
- Institutional mechanism

Part C: Investment Portfolio & Macro Economic Implications:

5. Economic & Financial Analysis

   5.1 Methodological Approach.
- Identification of cost and benefit
- Breakdown of local & foreign exchange components – 20% on irrigation sector – Foreign investment
- Use of dis counted approach & lost benefit analysis
- Treatment of price, transfer and subsidies and use of shadow pricing.
- Use of standard conversion factor
- Sustainability analysis
- Evaluation of benefits
- Treatment of target and regional balance
- Treatment of multipurpose projects.
- Estimated capital expenditure

   5.2 Sector approach in Analysis.

   a) Irrigation
- Surface – public goods, DIW – semipublic, STW – private

Cost borned- 3-5%,
15%
100%

STW installation under subsidy 5000/y & after cancellation of subsidy – 500/yr.

- Surface irrigation cost – 125 150 thousands/ha
- GW Irrigation – 125 20 thousand/ha
- Rs 5 billion cost ~ 175000 STW can cover entire terai
- Government spending Rs 4-5 billion in surface irrigation

b) Drinking Water

c) Hydropower and others

5.3 Portfolio Analysis

- Approach to identification of projects
- Project selection criteria
- Investment portfolio- public & private sector investment

5.4 project/program selection and investment

a) Investment – 1218.9 billion

9% - tenth plan
19% - Eleventh plan
21% - Twelveth plan
24% - Thierteenth plan
27% - Fourtheenth plan
Total investment hydropower -42%
Irrigation – 22%
Drinking water & sanitation – 19%
Electrification – 11%
Other – 5%
Irrigation sector – 266553 million
Structural – 261680 – Investment – 261680
O & M – 153540
Replacement – 74101
Agriculture support – 7341
Non Structural – 4872

b) Investment breaks down by sub- sector program

- Drinking Water
- Irrigation – Area Covered 600 thousand ha
- Agriculture support
- Hydro power
- Electrification
- Other use of water
- Fisheries
- Disaster management
- Environment management
- River basin planning
- Institutional development

5.5 Foreign exchange cost in water plan

5.6 Sectoral Return Analysis.

- Rate of Return Analysis

- Drinking water
- Irrigation – EIRR – 20, FIRR 15, BC- 1.28, 1.48
- Hydropower

5.7 Poverty Analysis, Regional balance & employment

5.8 Environmental aspects

5.9 Affordability

5.10 Financing mechanism for water sector development

a) Public & Private Sector Investment

61% -38.45% Total

85% -15% Irrigation

96% -4% Disaster prevention

b) Viable cost sharing mechanism

- Drinking water & Sanitation
- Irrigation
- Full cost recovery for STW
- Heavily subsidized DTW & surface irrigation
- Cost recovery (o & M) from users.
- Hydropower
- Institution
- Rural electrification
- Other water sub-sectors

5.11 International context.

- Large export of electricity

Part D: Environmental Plan

6. Environment Management Plan

- Introduction
- Implementation of mitigation plan
- Environmental components
    - Physical component
    - Biological
    - Socio-Economic component
- Environmental Monitoring
    - Monitoring
    - Types of monitoring – baseline, Impact, Compliance
- Environmental Auditing
    - Implementation of Audit
    - Institutional & procedural arrangement

Part E: Monitoring, Evaluation&Updating the plan

- Basic Framework for monitoring

- Basic Framework for Evaluation

२. सिचाई तथा
कृषि सम्बन्धि

२२०६०सिंचाइ नीति १.

१.१ पृष्ठभूमिः

- कृषि उत्पादन र उत्पादकत्व वृद्धिमा सिंचाइको अहम भुमिकालाई मध्यनजर राखी
- सिंचाइ पूर्वाधारको विकास र वस्तारलाइ निरन्तरता दिने वर्षै भरि सिंचाइ सुविधा उपलब्ध हुने क्षेत्र विस्तार गरी
- संगठित उपभोक्ताको प्रर्याप्त सहभागिता र लगानी वृद्धि

१.२ औचित्यः

- Conjunctive use and non-conventional irrigation
- Run off the river to year-round irrigation

१.३ उद्देश्यः

- देशमा विद्यमान जलस्रोतको प्रभावकारी उपयोग गरी सिंचाइ योग्य जमीनमा वर्षै भरी सिंचाइ सेवा विस्तार गर्ने
- विकसित प्रणालीको दिगो व्यवस्थापनका लागि उपभोक्ताको संस्थागत विकास गर्ने
- सिंचाइ क्षेत्रको विकाससंग सम्बन्धित प्राविधिक जनशक्ति उपभोक्ता र गैर सरकारी संघ।संस्थाको ज्ञान सिप र संस्थागत काई दक्षतामा अभिवृद्धि गर्ने

१.४ नीतिः

- सिंचित क्षेत्र घोषणा,
- एकीकृत जलस्रवत व्यवस्थापनको सिद्धान्त
- Year round irrigation
- Inter- basin water transfer
- जलाशययुक्त आयोजना
- भूमिगत जल भण्डारलाइ सतह सरह नै विकास
- निजी क्षेत्रलाइ संलग्न
- व्यवस्थापन हस्तान्तरण
- स्थानीय सरकार र उपभोक्ताको प्रभावकारी संलग्नता
- स्थानीय सरकार र उपभोक्ताको क्षमता अभिवृद्धि

- सिंचाइ सेवा सुविधाको परमाणात्मक मापन
- जनशक्तिको ज्ञान र सिपको विकास
- कानूनी र संस्थागत सुधार

भाग २ कार्यनीति

२.१. परिभाषा

- आयोजना, प्रणाली, मूल नहर, शाखा नहर, उपशाखा नहर, प्रशाखा नहर, कुलो, कुलेसो, बृहत आयोजना।प्रणाली, ठुलो आयोजनाप्रणाली, साना आयोजनाप्रणाली, परम्परागत सिंचाइ प्रणाली, उपभोक्तावाट संचालित प्रणाली, उपभोक्ता संस्था, संचालक, स्थानीय निकाय, एकीकृत सिंचाइ विकास

२.२ आयोजनको अध्ययन पहिचान र छनौट

- सामाजिक न्याय सन्तुलित विकास वातावरणीय सन्तुल वाली विविधिकरण व्यवसायीकरण र बढी आर्थिक प्रतिफल दिने Conjunctive Use को सम्भाव्यता समेत विचार गरी सरोकार संगको समन्वयनमा
- अन्तरजलाधार वहुउद्देशीय योजनाको अद्यावधिक सूची अनुसार कार्यान्वय
- Integrated River Basin Management को आधारमा स्थानीय निकाय र जनसमूदायको सहयोगमा
- संम्भाव्य योजनको अद्यावधिक सूची
- उपभोक्तालाइ छनौट चरण देखि नै संलग्न गराइ

२.३. आयोजना कार्यान्वयन कार्यविधि

- अनुमतिपत्र लिनु पर्ने
- माथिकी अग्राधिकार (Water Right) प्रत्याभूत
- एकीकृत कार्यान्वयन
- Use of Procedural Guidelines & Public Works Directive
- सरकारी र गैर सरकारी कार्यान्वयनमा एक रुपता

२.४उपभोक्ता समिति

- विभिन्न तहमा संगठित
- भौतिक निर्माणको क्रममै जउसको गठन र क्षमता अभिवृद्धि
- जउसमा ३३% महिला र दलित उत्पीडित र पिछडिएको जनजाती समूदायको सहभागित सुनिश्चित
- उपभोक्ता संस्थालाई जलाधार र राष्ट्रिय स्तरमा एकीकृत
- उपभोक्ता संस्था मार्फत जनसहभागिता

- एउटै निकायवाट नवीकरण र दर्ता

२.५ श्रोत परिचालन र जनसहभागिता

- उपभोक्ताको प्रत्यक्ष सहभागितामा हस्तान्तरण
- जमीन हस्तान्तरण उपभोक्ता संस्था मार्फत
- मालपोत मिन्हा र लगत कट्टाको व्वयस्था सम्बन्धित कार्यालयले
- Project Appraisalमा उपभोक्ताको लगानी अंश
- आयोजनना पहिचान सर्वेक्षण डिजाइन इष्टिमेट र निर्माणमा उपभोक्ता संस्थाको सहयोग
- हेभी इक्ववीप्मन्ट सःशुल्क उपलब्ध हुने
- स्थानीय वाधा हटाउने उपभोक्ता उपवलब्ध हुने
- संभाव्य श्रोतवाट आयोजनाको मर्मत सम्भार उपभोक्तावाट
- स्वदेशी तथा विदेशी लगानीमा प्रोत्साहन
- आकस्मिक मर्मत सम्भार कोषको स्थापना
- ISF को ८०% मर्मत सम्भार कझोष र २०% प्रशासनिक खर्च
- मर्मत सम्भार कोषको व्यवस्थापन सिंचाइ विभागवाट

२.६ प्रणाली व्यवस्थापन

- व्यवस्थापन प्रयोजनको लागि प्रणालीको वर्गीकरण

  क) उपभोक्तावाट संचालित परम्परागत सिंचाइ प्रणाली

  ख) सरकारी वा गैर सरकारी निकायवाट उपभोक्ता समितिलाई हस्तान्तरित

    नेपाल सरकारवाट संचालित

    नेपाल सरकार र जउसको संयुक्त व्यवस्थपनमा स्चालित

    स्थानीय नकिय र जउसको संयुक्त व्यवस्थापनमा संचालित

    नीजि स्तरवाट संचालित
- उपभोक्ताको माग अनुसार पुर्ननिर्माण।सुधार भएका र उपभोक्तावाट संचालित प्रणमालीको व्यवस्थपन जउसले नै गर्ने
- ३० हे भन्दा कम block को पूर्ण जिम्मेवारी जउसको हुने
- ५०० हे सम्मको block हस्तान्तरण गर्न सकिने
- पूर्ण व्यवस्थापन हस्तान्तरण मुल समितिलाई
- प्राविधिक रुपमा जटील प्रणालीफ संयुक्त व्यवस्थापन मार्फत
- संस्थागत विकास र आर्थिक क्षमता अभिवृद्धि मार्फत हस्तान्तरण
- कुनै व्यक्ति वा संस्थालाई प्रतिस्पर्दाको आधारमा कवुलियत गरि हस्तान्तरण
- नेपाल सरकारको व्यवस्थापन रहेका प्रणालीलाई क्रमिक रुपमा स्थानीय सरकारलाई हस्तान्तरण

- वहुउद्देशीय आयोनजना लगानी र प्रतिफलको आधारमा सिंचाइ विजुली र खानेपानीलेर्र् संचालन गर्ने
- आवश्यक पानी घटाएर मात्र अन्तर जलदाधार
- सिंचाइ संरचना र घोषित सिंचाइ क्षेत्र संरक्षण सिंचाइ विभागले गर्ने ।

२.७ सिंचाइ सेवा शुल्क र आयस्रोतको अन्य उपाय

- सिंचाइ व्यवस्थापनवाट वठेको आम्दानीको आधारमा ISF
- ISF नवुझाए सेवावाट वञ्चित हुने
- ISF साधारण सभावाट पारित
- सिंचित क्षेत्रको अभिलेख उपलब्ध गराउनु पर्ने
- ISF उठाउने दायित्व WUA को

२.८ मर्मत सम्भार र प्रणाली संचालन

- समानुपातिक हिसावले पानी वाँडफाँड
- हस्तान्तरित योजनाको Regular maintenance WUA वजाट
- Technical support सिंचाइ विभागवाट
- सम्झौता अनुरुप संयुक्त व्यवस्थापन
- ISF ले नपुगेमा थप शुल्क उठाउन सक्ने

२.९ व्यक्ति समूह वा गैर सरकारी संघ।संस्था

- निजी क्षेत्रले सशुल्क निर्मपाण वा संचालन गर्ने व्यवस्था
- कृषकवाट संचालितलाई पूर्ण स्वतन्त्रताको प्रत्याभूति

२.१० जवाफदेही र जिम्मेवारी

- सिंचाइ विभाग र जउस वीच औपचारिक सम्झौता पछि आयोजनामा लगानी
- क्षतिपूर्ति माग गर्न सक्ने जउसले
- जउसले सम्झौता वमोजिम काम नगरेमा रकम रोक्का
- वृहत ठुला तथा मझौला आयोजनाको जिम्मेवारी सिंचाइ विभागको

२.११ वातावरण संरक्षण एवं जल गुणस्तर

- वातावरणीय नकारात्मक प्रभाव न्यून हुने गरी कार्यान्वयन
- Public Hearing सहित IEE or EIA गर्ते
- Ecological water छोडेर जैविक विविधताको संरक्षण
- अनुगमन अध्ययन र अनुसन्धान गर्ने
- Environmental mitigation and protection work लागत अनुमानमा संलग्न गर्ने

२.१२ प्रविधि विकास तथा प्राविधिक जनशक्ति

- सिमान्तकृत भूमिका लागि NITP
- औपचारिक शिक्षा प्रणालीमा सिंचाइ
- Capacity building
- Data base तयार गर्ने
- वजेट र जनशक्ति

२.१३ अन्य निकायसंगको समन्वयन

- सिंचाइ संरचनको चर्चेको सम्पतिको जिम्मेवारी र जवाफदेही प्रचलित कानून वमोजिम
- गैर सरकारीको संलग्नता वठाइने नीजी क्षेत्रको संस्थागत विकास गर्ने
- जल तथा मौसम विज्ञान संग समन्वयन
- कृषि कार्यक्रम संग आवद्ध
- Priority package program संग आवद्ध

२.१४ अनुगमन तथा मूल्याङ्कन

- केन्द्रीय अनुगमन समिति
- Monitoring&Evaluationप्रणालीलाई संस्थागत गर्ने
- पानीको परीमाण वाली सघनता उत्पादखन वृद्धि उपभोक्ता संस्थाको आर्थिक स्थिति र कार्यक्षेत्रको आधारमा अनुगमन मूल्याङ्कन

२.१५ विविध

- भूमिगत जलस्रोतको संरक्षण
- विधुतमा विशेष सहुलियत
- उत्कृष्ठ उपभोक्ता संस्थालाइ बार्षिक रुपमा पुरस्कृत
- सिंचाइ विकास कोषको स्थापना
- Documentation Centre and Water Use Status Preparation.
- विदेशी परामर्शदाताको न्यून संलग्नता रहेको योजनालाई प्राथमिकता
- भारी उपकरण आयातमा छुट
- हरेक पाँच वर्षमा अद्यावधिक

अनुसूचि १

आयोजनामा उपभोक्ताले व्यहोर्नुपर्ने अंशको गणना तालिका

सिंचित क्षेत्र	Head Works	Main Canal	Branch Canal	Tertiary Canal
20.5 ha	0	0	0	10
0.5 – 1.0 ha	0	0	5	10
1-5 ha	1	3	7	12
>5 ha	3	5	10	15

अनुसूचि २

कृषक उपभोक्ताहरुवाट उठ्ने सिंचाइ सेवा शुल्कको वाँडफाँड सम्बन्धि व्यवस्था

	Central Fund	Revenue	WUA
1 Canal WUA other GoN	40	40	20
2 After tertiary WUA	30	30	40
3 After Branch WUA	20	20	60
4 After Main Canal	10	10	80
5 Except headworks	5	5	90
6 all system transferred	0	5	95

२सिंचाइ नियमावली २., २०५६

जलस्रोत ऐन २०४९ को दफा २४ प्रयोग गरी

परिच्छेद १ प्रारम्भिक

१. संक्षिप्त नाम र प्रारम्भ २०५६ तुरुन्त

२. परिभाषा

- संरचना
- उपभोक्ता संस्था
- सेवा
- कार्य समिति
- परियोजना
- परियोजना समिति
- सेवा शुल्क
- विभाग
- परियोजना कार्यालय
- सिंचाइ प्रणाली
- बृहत सिंचाइ आयोजना
- ठूला सिंचाइ आयोजना
- बहुउद्देशीय सिंचाइ आयोजना
- मझौला सिंचाइ आयोजना
- साना सिंचाइ आयोजना
- परम्परागत सिंचाइ प्रणाली
- नयाँ प्रविधिमा आधारित सिंचाइ प्रणाली
- लिफ्ट सिंचाइ प्रणाली
- भूमिगत जल सिंचाइ प्रणाली

परिच्छेद २ उपभोक्ता संस्था तथा परियोजना हस्तान्तरण सम्बन्धि व्यवस्थाः

३. उपभोक्ता संस्थाको दर्ता

- ३३% महिला दुइ जना दलित उत्पीडित र पिछडिएको जनजाति सहित ११ जना
- सिंचाइ कार्यालयमा दरखास्त
- ६७ प्रतिशत उपभोक्ताको प्रतिनिधित्व
- दर्ता गरी प्रमाणपत्र दिनु पर्ने

४. उपभोक्ता संस्थाको निर्वाचन तथा कार्य समिति विघटन

- संस्थाको विधान वमोजिम
- २।३ साधारण सदस्यको निर्णयले विघटन
- नियमावली वा विधान विपरीत कार्य गरेमा १५ दिने कारण माग सिंचाइ कार्यालयले
- सन्तोषजनक जवाफ नआएमा विघटनको सिफारिस
- सिंचाइ विभागको सहमतिमा सिंचाइ कार्यालयले विघटन गर्न
- म्याद भित्र नवीकरण नभएका विघटन
- नयाँ निर्वाचन नभए सम्म कुनै सुविधा उपलब्ध नगराउने
- सिंचाइ कार्यालयले जिल्ला सिंचाइ महासंघको रोहवरमा चुनाव

५. उपभोक्ता संस्थाको काम कर्तव्य र अधिकार
- मर्मत सम्भार संचालन तथा व्यवस्थापन
- पानीको वितरण
- सिंचाइ सेवा पुर्याउन नसकेको लगत र मिनाहा
- पहिलेका असर नपर्ने गरी थप उपभोक्तालाई सुविधा
- जनसहभागिता परिचालन
- सिंचाइ सेवा वृद्धि
- ISF Collection
- ISF नवुझाउनेलाई सेवावाट वञ्चित
- विगार गर्नेको सूचना सिंचाइ कार्यालयलाई
- Technical Support by सिंचाइ कार्यालयलाई
- उप समिति वनाई अधिकारी प्रत्यायोजन

६. अभिलेख राख्नु पर्ने
- कोषको अभिलेख
- बर्ष समाप्त भएको तिन महिनामा सिंचाइ कार्यालयलाई विवरण वुझाउने

७. उपभोक्ता समन्वयन संस्था
- समन्वयन संस्थाको गठन गरी जिल्ला सिंचाइ उपभोक्ता महासंघ र सम्वन्धित सिंचाइ कार्यालयलाई दिने

८. उपभोक्ता समन्वयन संस्थाको दर्ता
- नियम ३ वमोजिम

८क. नवीकरण
- महालेखावाट लेखा परीक्षण (९० दिन भित्र) गरी नवीकरण गर्नुपर्ने
- निलम्वन शुल्क १०० थप नव्वे दिन
- ७ दिन भित्र नवीकरण

९. मर्मत सम्भार कोष

- आयको नव्वे प्रतिशतको मर्मत सम्भार कोष

१०. परियोजना हस्तान्तरण

- सरकारले उपभोक्ता संस्थालाई

११. परियोजना हस्तान्तरणका शर्तहरु

- धितो वन्धक व्चविखन दान सट्टापट्टा गर्न नपाउने
- विगार्न नास्न अदलबदल गर्न नपाउने
- पानीक कमी र गुणस्तरमा ह्रास आउन नपाउने
- उपभोक्ताले उपयोग गरेको पानी कम गर्न नपाउने
- अरु व्यक्ति र संस्थालाई संचालनको जिम्मा दिन नपाउने

१२. बृक्षारोपण तथा रुख विरुवाको संरक्षण

- Right of Way मा WUA ले वन ऐन वमोजिम
- कार्ययोजना स्वीकृत नभएको अवस्थामा तपसीलको समिति
- सिंचाइ कार्यालयको प्रमुख – संयोजक
- जिविस प्रतिनिधि – सदस्य
- जिल्ला प्रशासन प्रतिनिधि – सदस्य
- वन कार्यालय प्रतिनिधि – सदस्य
- WUA अध्यक्ष – सदस्य
- जिल्ला सिंचाइ उपभोक्ताका सवै प्रतिनिधि– सदस्य

१३. संयुक्त व्यवस्थापन प्रणाली

- पूर्ण हस्तान्तरण हुन नसकेको हकमा संयुक्त व्यवस्थापन

परिच्छेद ३ खारेजः

परिच्छेद ४ सेवा उपयोग सम्वन्धी व्यवस्था

१८. दरखास्त दिनु पर्ने

- सेवा उपभोगको लागि दरखास्त दिनु पर्ने
- प्राविधिक जाँच गरी सेवा उपलब्ध गराउने र सेवा उपलब्ध गराउन नसकिने भए जानकारी दिने

१९. उजुरी दिन सकिने

- सेवा उसपलब्ध गराउने नसकिने जानकारी चित्र नवुझे ३५ दिन भित्र उजुरी गर्ने
- जाँचवूझ गरी सिंचाइ कार्यालयवाट हुने निर्णय अन्तिम हुने

२०. सेवा उपभोषगको शर्तः

- अनुमति पत्र प्राप्तले तोके वमोजिम

२१. सेवा उपलब्ध गराउने आधार

- सम्बन्धित क्षेत्रको भौगोलिक स्थिति
- जग्गाको क्षेत्रफल
- श्रोत उपलब्ध हुन सक्ने पानीको परिमाण
- जग्गामा लाउने वालीकि किसिम
- जग्गाको माटोको प्रकृति
- संरुचनाको क्षमता र अन्य प्राविधिक कुरा

२२. सेवा कटौती गर्न सक्ने

- पानीको श्रोतमा कमी वा संरचनाको क्षमतामा कमी आए

२३. सेवा वन्द गर्न सक्ने

- ISF नबूझाएमा
- शर्त उल्लंघन गरेमा
- संरचना क्षतिग्रस्त भएमा वा हुने सम्भावना भएमा आवश्यक मर्मत सम्भार नगरे सम्म

२४. सूचना दिनु पर्ने

- सेवा कटौती वा वन्दको

परिच्छेद ५ उपभोक्ताको कर्तव्य र दायित्व तथा सेवा शुल्क सम्बन्धि व्यवस्था

२५. उपभोक्ताको कर्तव्य र दायित्व

- सेवा दुरुपयोग चुहावट तत्कालै जानकारी दिने
- निर्माण मर्मत सम्भार र सुरक्षामा कार्यालयलाई सहयोग गर्ने

२६. सेवा शुल्क निर्धारण समितिको गठन

- प्रमुख सिंचाइ कार्यालय – अध्यक्ष

- प्रतिनिधि जिल्ला कृषि विकास कार्यालय – सदस्य
- जउसको अध्यक्ष – सदस्य
- जिल्ला सिं जल उपभोक्ता संघको प्रतिनिधि – सदस्य

२७. सेवा शुल्क निर्धारण समितिको बैठक
- अध्यक्षले तोकेको स्थान र मितिमा
- कम्तीमा बर्षको पएक पटक
- कार्यविधि आफैले तोके वमोजिम

२८. सेवा शुल्क निर्धारण समितिको काम कर्तव्य र अधिकार
- वठ्ने कृषि उत्पादनको आधारमा सेवा शुल्क निर्धारण
- नेपाल सरकारले मागेको विवरण र परामर्श उपलब्ध गराउने
- सेवा शुल्कको दर हेरफेर

२९. सेवा शुल्क वुझाउनु पर्ने
- उपभोक्ताले सेवा शुल्क वुझाई रसिद बुझ्ने
- संयुक्त व्यवस्थापन प्रणालीको सेवा शुल्क उठाउने जउसले
- सेवा शुल्क कोष राजश्व र जउसको बाँडफाँड नियम अनुसार
- केन्द्रीय मर्मत सम्भार कोष र राजश्वमा जग्गा नभएका सिंचाइ प्रणालीमा थप लगानी गर्ने छैन ।

३०. विलम्व शुल्क
- निर्धारित समयमा शुल्क नबुझाउने

परिच्छेद ६ सिंचाइ परियोजना सम्बन्धी व्यवस्था

३१. ठुलो प्रकृतिका सिंचाइ परियोजनाहरुका लागि

सचिव – सिंचाइ

मन्त्रालय – अध्यक्ष

महानिर्देशक – सिंचाइ

विभाग – सदस्य

महानिर्देशक –

कृषि

विभाग –

सदस्य

प्रतिनिध –

अर्थ

मन्त्रालय –

सदस्य

प्रतिनिध –

रायोआ

–

सदस्य

प्रतिनिध –

सामान्य

प्रशासन मन्त्रालय –

सदस्य

सम्बन्धित –

परियोजना

प्रमुख –

सदस्य

सचिव

३२. परियोजना समितिको वैठक

- अध्यक्षले तोकेको मिति समय र स्थान
- अध्यक्षले अध्यक्षतामा अध्यक्ष नभए समितिले छानेको व्यक्तिले
- पचास प्रतिशत गणपुरक संख्या
- वहुमतको निर्णय
- सदस्य सचिवले प्रमाणित
- अन्य कार्य विधि तोके अनुसार

३३. परियोजना समितिको काम कर्तव्य र अधिकार

- परियोजना समयमा सम्पन्न गराउने
- परयोजना सम्बद्ध डिजाइन ड्रईङ्ग आदि प्रावित्रधिक काम गराउने
- कर्मचारीको दरवन्दी स्वीकृत गर्ने

- परियोजना सम्पन्न गर्ने अरु काई

३४. परियोजना प्रमुखको नियुक्ति

- नेपाल सरकारले

३५. परियोजनाको कर्मचारी सम्बन्धी व्यवस्था

- आवश्यक कर्मचारी अस्थायी रुपमा स्वीकृत गर्ने

३६. अधिकार प्रत्यायोजन

- परियोजना प्रमुखलाई र प्रमुखले मातहतका कर्मचारीलाई

३७. परियोजनाको दायित्व सर्ने

- विभागले तोकेको कार्यालयलाई

३८. परियोजना इकाई कार्यालय

- दुई वा दुई भन्दा बढी जिल्लामा विस्तारित परियोजनाको

परिच्छेद ७ विविध

३९. संरचनाको सुरक्षा व्यवस्था

- संरचनामा अनाधिकृत प्रवेश
- संरचना भत्काउन वन्द गर्न वा हेरफेर गर्न
- संरचनामा आउने पानी घटाउन वा वढाउन
- संरचनाको पानी प्रदुषण गर्ने
- संकेत वा चिन्ह विगार्न वा सार्न
- यन्त्र वा पार्टपुर्जा सार्न वा विगार्न
- चौपाया हिढाउन चर्न वा छाडा छाड्न
- प्रवेश निषेध क्षेत्रमा सवारि चलाउन
- वाँध वा नदीको ढुँगा वालुवा आदि निकाल्न
- नहरको डिल काट्नज नहर वा संरचना विगार्न
- नदी वा खोलाको धार अवरोध गर्दा
- सिंचाइ व्यवस्थामा प्रतिकुल असर गर्न
- स्वीकृति वेगर पम्प चलाउँदा

40. दण्ड सजाय

दिने अधिकारी

- सिंचाइ कार्यालयको प्रमुख

41. निर्देशन

दिन सक्ने

- निर्माण गरेको समितिले

42. जनसहभागिता

- नियम अनुसार
- ३-१५%

४३. अनुगमन तथा मूल्याङ्कन समिति

- नेपाल सरकारले आवशेयकता अनुसार

४४क. व्यवस्थापन हस्तान्तरण गर्न सकिने

- स्थानीय निकायलाई

४४. मूल्याङ्कन तथा गुणस्तर कायम

- क्षेत्रीय निर्देशक मार्फत

४५. निर्देशिका वनाउन सक्ने

- नेपाल सरकारले

४६. अनुसूचीमा थप घट वा हेरफेर गर्न सक्ने

- नेपाल सरकारले

४७. खारेजी र बचाउ

- सिंचाइ नियमावली २०४५ खारेज

1.8 चालु आवधिक योजना (पन्ध्रौ योजना)

आव २०७६।७७ - २०८०।८१

परिच्छेद १ परिचयः

- अन्तराष्ट्रिय प्रतवद्धता अनुरुप विस २०८७ सम्म दिगो विकास लक्ष्य हासिल गर्नुपर्ने
- विस २०६९ सम्ममा नेपाललाई अतिकम विकसित देशबाट विकासशील देशमा स्तरोन्ति गर्दै विस २०८७ सम्ममा मध्यम आय भएको मुलुकको स्तरमा पुर्याउने
- हालसम्म नौ वटा पञ्चवर्षिय र पाँच वटा त्रिवर्षिय योजना कार्यान्वयन
- २०७२ को पभुकम्पले अर्थतन्त्रमा करिव सात खर्च वरावरको क्षति
- आव २०७५।७६ सम्ममा अर्थतन्त्रको आकार रु ३४ खर्ब ६४ अर्ब
- चौधो योजनाको लक्ष्य र उपलब्धि

चक	आधार बर्ष (२०७२।७३)	.ब. २०७५।७६ सम्मको लक्ष्य	.ब. २०७४।७५ को प्रगति
आर्थीकि वृद्धिदर	।८	।२	।९
कृषिक्षेत्रको वृद्धिदर	।३	।७	।२
गैर कृषि क्षेत्रको वृद्धिदर	।६	।४	।०
गरीवीको रेखामुखीको जलसंख्या	१।६	७	८।७
सिंचाइ (हेक्टर लाखमा)	३।९	५।२	४।७

- सिंचाइमा उपेक्षित उपलब्धी हासिल हुन नसक्नपु – भौगोलिक विकटता जमीनको असहज अवस्थित जल स्तथानान्तरण तथा जलाशययुक्त बहुउद्देशीय इजना अगी वढ्न नसक्नु प्राकृतिक प्रकोपबाट सिंचाइमा क्षति
- दिगो विकासको लक्ष्य – २०१६ – २०३० सन्
- १७ वटा लक्ष्य १६९ परिमाणात्मक लक्ष्य र २३२ वटा विश्वव्यापी सूचक
- दिगो विकास लक्ष्य हासिल गर्न
  - बार्षिक औषत ➧ २० खर्ब २५ अर्ब ➧ ११ खर्ब ११ अर्ब सरकारी क्षेत्रबाट ➧ ७ खर्ब ३९ अर्ब निजि क्षेत्रबाट ➧८७ अर्ब सहकारी र गैरसरकारी क्षेत्रवाट ➧ ८८ अर्ब घर परिवार क्षेत्रवाट

परिच्छेद२दीर्घकालिनसोच २१००

- समृद्धि र सुख प्रासिका तीन चरणः
- पहिलो चरण समृद्धि र सुखको आधार निर्माणको चरण।यस चरणमा आर्थिक वृद्धिका लागि आर्थक समाजिक र भौतिक पूर्वाधार तयार गर्न । पन्ध्रौ योजना प्रथम चरण समृद्धि र सुखको आधारशिला तयार गर्न ।
- दोश्रो चरण समृद्धि र सुखका सूचकहरुमा तीव्र प्रगति हासिल गर्ने चरण । दुइ पञ्चवर्षिय योजनाको चरण ।
- तेश्रो चरण समृद्धि र सुखको सूचकहरुमा सन्तुलन सहितको दिगोपना हासिल गर्न चरण । दुइ पञ्चवर्षिय योजना
- दीर्घकालिन राष्ट्रिय लक्ष्य

Prosperity:

- Accessible modern infrastructure and intensive connectivity
- Development and full utilization of human capital potentials
- High and sustainable production and productivity
- High and equitable national income

Happiness:

- Well being and decent life
- Safe, civilized and just society
- Healthy and balanced environment
- Good governance
- Comprehensive democracy
- National unity, security and dignity

- दीर्घकालिन सोचका प्रमुख लक्ष्य

	२०७४।७५	२१००।०१
आर्थिक बृद्धिदर	६।८	१०।५
कुल गार्हस्थमा कृषि र बनक्षेत्रको योगदान	२७।०	९।०
प्रतिव्यक्ति कुल राष्ट्रिय आय	१०४७	१२१००
जलविधुत तथा नवीकरणीय ऊर्जा उत्पापदन	१०७४	४००००

- यस अवधिमा २२ लाख हेक्टर कृषि योग्य भूमिमा भरपर्दो सिंचाइ व्यवस्था पुर्याउने ।
- प्रमुख नदी किनारमा बाढी पहिरो नियन्त्रण र व्यवस्थापन गर्ने
- ३५ हजार जलविधुत र ५ हजार नवीकरणीय ऊर्जा बाट विधुत

परिच्छेद ३ पन्ध्रौ योजना (२०७६।७७ - २०८०।८१)

		३२०७४।७५	२०८०।८१
आर्थिक बृद्धिदर	प्रश	६।८	१०।३
प्रतिव्यक्ति कुल आय	अ ड US$	१०४७	१५९५
निरपेक्ष गरिब	प्रश	१८।७	११
कृषि वन र खानी क्षेत्रो अर्थतन्त्रमा योगदा	प्रश	२७।६	२३।०
श्रम उत्पाकत्व	रु हजारमा	१८४।६	२७६
कृषि उत्पाकत्व	Mt/ha	२।९७	४।०

- सिंचाइ सुविधाको तीव्र र सघन विस्तार भूउपोग नीतिको प्रभावकारी कार्यान्वयन जमीनको एकीकरण तथा चक्लावन्दी उन्नत वीउ वीजन मल तथा आधुनिक प्रविधिको अवलम्वन कम तौल।आयतन उच्च मूल्य तथा अर्गानिक कृषि उत्पादन र प्रशोधनमा जोड दिंदै कृषिको यान्त्रिकरण आधुनिकीकरण व्यवसायीकीकरण र औधोगिकीकरण गरिनेछ ।

परिच्छेद ४ समष्टिगत अर्थतन्त्र

४.१ वचन

४.२ सार्वजनिक वित

४३ सार्वजनिक संस्था

४।४ मौद्रिक तथा वितीय क्षेत्र

४।५ मूल्य नीति

४।६ वैदेशिक व्यापार शोधनान्तर र विदेशी विनियम

विमा

४।७ विभा

४।८ पुँजी बजार

परच्छेद ५ नीजि तथा सहकारी क्षेत्रः

५।१ निजी क्षेत्र

५।२ सहकारी क्षेत्र

५।३ सार्वजनिक निजी क्षेत्र

५।४ वैदेशिक लगानी

५।५ विप्रेषण

५।६ उद्यमशिलता विकास

परच्छेद ६ आर्थिक क्षेत्र

६।१ कृषि तथा प्राविधिक स्रोत

६।१।१ कृषि

- कुल ग्राहस्थ्य उत्पादनमा ~ २७% योगदान ➧ जनसंख्याको आवद्धता ~ ६०।४%
- नेपालको संविधान खाद्य सम्वन्धि अधिकार
- कृषि विकास रणनीति (२०१५ – २०३५)र दिगो विकास लक्ष्यलाई मार्गदर्शन
- प्रमुख समस्या

- कृषि उत्पाकत्व वृद्धि गर्न अपरिहार्य स्रोत साधन र सामाग्रीहरुको न्युन उपलब्धता आवश्यक भौतिक पूर्वाधारहरु जस्तै सिंचाइ सडक कृषि सडक शीत भण्डार गोदाम घर चिस्यान केन्द्र कंकलन केन्द्र तथा विजुली आदिको अपर्याप्ता
- उन्नत नश्ल तथा वीऊको प्रतिस्थापना दर अत्यन्तै कम
- अतिरिक्त जमीनकोप तीव्र खण्डीकरण प्रमुख समस्या

- अनुसन्धानवाट विकास भएका प्रविध प्रर्याप्त मात्रामा विस्तार नहुनु वैज्ञानिक जनशक्ति व्यवस्थापन कमजोर हुनु अनुसन्धानको लागि प्रयोगशाला लगायत अन्य पूर्वाधारहरुकझो कमी हुनुले कृषिको आधुनिकीकरण यान्त्रिकीकरण व्यवसायीकरण तथा औधोगिकरणमा वाधा
- उत्पादित खाद्यवस्तुहपरु पनि समुचित भण्डारण प्रशोधन तथगा मुल्य अभिवृद्धि गरी प्रतिफल यथोचित रुपमा वढाउन नसकिएको
- जलवायु परिवर्तनवाट यस क्षेत्रमा पर्न गएको प्रभावलाई कम गर्न अनुकुलनको माध्यमवाट जलवायु उत्थानशील तथा वातावरण मैत्री कृषि प्रणालीको विकासमा समस्या

६१।२ खाद्य सुरक्षा तथा पोषणः

६।१।३ सिंचाइ

- कृषि योग्य जमिन – २६ लाख ४१ हजार

सिंचाइ योग्य जमिन – २२ लाख ६५ हजार

२०७४।७५ सम्म – १४ लाख ७३ हजार पूर्वाधार विकास

सोच – दिगो एवं भरपर्दो सिंचाइ सुविधा उपल्ध ग९राई कृषि उत्पादन र उत्पाकत्व वृद्धिमा योगदा

लक्ष्यः – कृषि योग्य भूमिमा दिगो एवं भरपर्दो सिंचाइ सुविधा उपलब्ध गराउने

उद्देश्यः– उपर्युक्त प्रविधि थप कृषि योग्य भूमिमा सिंचाइ सेवा विस्तार गर्नु

- ठूला जलाशययुक्त तथा जलस्थानान्तरण बहुउद्देशीय आयोजनाको विकास गरी कृषि योग्य भूमिमा बर्षै भरी भरपर्दो रुपमा सिंचाइ सुविधा उपलब्ध गराउनु
- सम्पन्न सिंचाइ प्रणालीको मर्मत सम्भार एवं व्यवस्थापनलाई सुदृढ तुल्याई दिगोपन वढाउनु

रणनीति तथा कार्यनीति

१. सिंचाइविकासकोगुरुयोजनातथाकृषिविकासरणनीतिअनुसारजलवायु परिवर्तन अनुकूलन हुने गरी सिंचाइ योजनाहरुको विकास एवं विस्तार गर्ने ।
२. नयाँ प्रविधिमा आधारित सिंचाइको विकास गर्नुका साथै सिंचाइ दक्षता वृद्धि गर्ने ।

३. संघ प्रदेश र स्थानीय तहको समन्वयन र सहकार्यमा सिंचाइ प्रणालीको विकास गर्न तथा बाह्रै महिना सिंचाइ सेवा उपलब्ध गराउनका लागि ठूला बहुउद्देशीय अन्तर जलाधार र जलाशययुक्त आयोजनालाई प्राथमिकताका साथ अघि वढाउने
४. भूमिगत सिंचाइ योजनाको विस्तार सहित उपयोगमा जोड दिने
५. सिंचाइ प्रणालीको मर्मत सम्भार एवं दिगो व्यवस्थापनका लागि स्रोत सहित उपभोक्ता सहभागिता सुनिश्चित गर्ने ।
६. नीतिगत सुधार एवं विद्यमान संस्थागत संरचनाको क्षमता र जनशक्तिको दक्षता अभिवृद्धि गर्ने ।

अपेक्षित उपलब्धीः

- थप ३ लाख हेक्टरमा सिंचाइ सुविधा
- ५०% क्षेत्रमा बर्षै भरी सिंचाइ सुविधा
- ९८५०० हेक्टर भूमिमा सिंचाइ सेवा पुगेको क्षेत्रमा प्रभावकारी व्यवस्थापन

Water Allocation and the Human Right to Water

Millennium Goal 7 and Sustainable Development Goal 6

Water According to the United Nations Children's Fund (UNICEF) and the World Health Organization (WHO 2019), more than two billion people in the world did not have access to safe drinking water, and another two billion people lacked access to basic sanitation in 2019. In 2010, the UN General Assembly declared the access to water, be it as drinking water or a medium for sanitation and hygiene, as a human right. Together with six additional goals, which range from halving the proportion of people living in extreme poverty to reducing the under-five mortality rate by two-thirds between 1990 and 2015, the Millennium Goal 7 called to

Halve, by 2015, the proportion of the population without sustainable access to safe drinking water and basic sanitation.

In 2015, these Millennium Development Goals were replaced by the Sustainable Development Goals consisting of 17 goals ranging from poverty and hunger eradication to strategies aiming at building peaceful and inclusive institutions. Goal 6 refers to clean water and sanitation, according to which universal access to safe and affordable drinking water should be ensured by 2030—quite an ambitious goal in the face of climate change leading to water scarcity, specifically in those areas of the world with the poorest inhabitants.

From the perspective of IWRM, the achievement of Goal 6 requires to tackle the access problem and, at the same time, to protect the catchment areas against an overutilization of water. Water scarcity translates into high water prices, which in turn brings about an optimal allocation of water use. This approach will only result in an optimal equilibrium if all market participants can afford the amount of water to cover their basic needs for a secure conduct of life. There is a broad literature on basic water needs, the lower range of which would be in the range of 15 liters per day and capita (lpd) (Reed et al. 2011).

In addition to water, households need a certain daily endowment of calories and nutrition as well. Therefore, poor households need a minimum income to survive in order to finance expenses that allow them to buy the subsistence basket of basic goods, containing water, food (nutrition), housing, and shelter. But often poor households do not earn enough money to secure this lifeline. It is rather obvious that price increases can affect these households in a very detrimental way. We, therefore, cannot trust in unregulated markets as institutions that secure efficiency. Classical welfare theory assumes that a market participant can make a living based on her income. Hence, the demand for goods is solely the expression of preferences following from taste and predispositions. In the case of poor households, we cannot assume that their demand for basic goods is the result of optimizing their demand according to these kinds of preferences. Often, the demand for goods is nothing else than the result of poverty management below the lifeline. The composition of food purchased is optimized with respect to calorie content. Hence, in this case, revealed preferences are based on survival strategies and not on taste.

This view coincides with social-psychological theories of need management. The famous Maslowian need hierarchy describes the stratification of human needs whose satisfaction is expressed in corresponding actions be it the demand for water and nutrition or supply of labor. At the bottom is the satisfaction of physiological needs, followed by other needs such as security and social recognition. In our case, the satisfaction of physiological needs is essential. Needs at this level are undoubtedly legitimized by human rights. If markets do not guarantee their satisfaction the welfare theoretical criterion of efficiency or social optimality is irrelevant.

Take as an example the Pareto criterion economists often refer to: A reallocation of goods is said to be socially preferable to a given distribution of goods if it increases the welfare of one or more members of society without harming the well-being of others. This approach might be suitable for a middle-class society but not for an economy divided into poor people and members endowed with sufficient financial means to not only satisfy basis needs but also to buy those products and services which allow individual self-fulfillment at a higher level of the hierarchy of needs. For instance, increasing the welfare of the latter group by allowing good exchange between both classes does not increase social welfare. Here we want to refer to the Rawlsian social welfare function where social welfare depends solely on the well-being of the poorest.

IWRM has to take into account this distinction between taste-driven consumer choices and revealed purchase behavior resulting from survival strategies. In this sense, poor households have to be included in the IWRM models that deliver allocation mechanisms that guarantee the subsistence level of drinking water, sanitation, and other basic goods and services in line with the Sustainable Development Goals.

1.3 History of irrigation development in Nepal

देशमा सिंचाइ सुविधा विकास र विस्तारको क्षेत्रमा परापूर्व काल देखि कृषकहरु आफैले र खास गरी योजनावद्ध विकासको थालनी उप्रान्त नेपाल सरकारले लगानी गर्दै आईरहेको छ । सरकारी स्तरबाट वि. सं. १९७९ देखि चन्द्र नहरको निर्माण शुरु भए पश्चात मात्र सिंचाइको विकास शुरु हुन गएको देखिन्छ । तत्पश्चात वि.सं. १९८५ मा निर्मित चन्द्रनहरबाट पानी वितरण शुरु भई देशमा आधुनिक सिंचाइ प्रविधिको प्रादुर्भाव हुन गयो । वि.सं. २००० सालतिर सर्लाहीमा जुद्ध नहरको निर्माण गरियो भने कपिलवस्तुमा जगदीशपुर वाँध र पोखरामा पार्दी वाँधको थालनी गरियो । वि.सं. २००७ साल भन्दा अगाडिसम्म सिंचाइ क्षेत्रमा सरकारी संलग्नता सिमित थियो । सिंचाइ विकासका कार्यहरु

बडाहाकिम मार्फत कार्यान्वयन हुने गर्दथे भने स्वदेशी प्राविधिक जनशक्तिको अभावमा विदेशी जनशक्ति समेत प्रयोग गर्नु पर्ने अवस्था थियो । वि.सं.२००७ सालको प्रजातान्त्रिक परिवर्तन पश्चात् स्वदेशी प्राविधिक जनशक्तिको उत्पादन शुरु भएतापनि वि. सं. २००९ सालमा स्थापना भएको नहर विभागको नेतृत्वको जिम्मेवारी वि. सं. २०१३ साल देखि मात्र नेपाली प्राविधिकबाट भयो । २०१३ सालमा प्रथम पञ्चवर्षीय योजना लागु भएपछि योजनाबद्ध रुपले सरकारी स्तरबाट सिंचाइ विकासको थालनी भएको हो । कतिपय आयोजनाहरुमा विकास समिति गठन भई कार्यान्वयन गर्न थालियो भने कतिपय केन्द्रीय स्तरमा नै आयोजना कार्यान्वयन गरिए । निर्माण सम्पन्न आयोजनाहरुमा आवश्यकता अनुरुप डिभिजन÷सव डिभिजन स्थापना गरी संचालन र मर्मत संभार गर्न थालियो । २०२८ सालमा क्षेत्रगत हिसाबले ४ वटा क्षेत्रीय निर्देशनालय स्थापना गरियो भने २०३७ सालमा क्षेत्रीय निर्देशनालयको संख्या ४ वटा बाट ५ वटा पुऱ्याइयो । सिंचाइ संस्थागत विकासका क्रममा नहर विभाग, सिंचाइ तथा खानेपानी विभाग, सिंचाइ तथा जलवायु विज्ञान विभाग हुँदै २०४४ सालमा सिंचाइ विभागको नामाकरण गरि समग्र सिंचाइ विकास तथा नदी नियन्त्रणको प्रतिनिधि संस्थाको रुपमा स्थापित हुन पुग्यो । बि.सं. २०४५ साल देखि सिंचाइ बिभाग अन्तर्गत ५ क्षेत्रिय निर्देशनालय, ७५ जिल्ला सिंचाइ कार्यालय, र केन्द्रिय आयोजनाहरु संचालनमा आए । तत्त पश्चात स्थानीय स्वायत शासन ऐन, २०५६ जारी भइ लागु भए पछि वि.सं. २०५८ मा सिंचाइ विभागको संरचनात्मक फेरबदल भयो । सोहि क्रममा विभाग अन्तर्गत रहेको जल उत्पन्न प्रकोप नियन्त्रण प्रविधि केन्द्र (Disaster Prevention Technical Centre) को रुपमा स्थापना भै जल उत्पन्न प्रकोप नियन्त्रणको क्षेत्रमा विभिन्न कार्यक्रमहरु शुरु गरेकोमा यसै केन्द्रलाई त्यसको उद्देश्य तथा उपलब्धिहरुलाई संस्थागत गर्न स्थायी संरचना तयार गर्ने क्रममा २०५६ माघ २४ गते जल उत्पन्न प्रकोप नियन्त्रण विभागको स्थापना गरियो । बि.सं. २०५८ मा सिंचाइ बिभाग अन्तर्गत ५ क्षेत्रिय निर्देशनालय, २६ सिंचाइ बिकास डिभिजन, २० सिंचाइ बिकास सव डिभिजन, ८ सिंचाइ ब्यवस्थापन डिभिजन, ३ यान्त्रिक डिभिजन तथा केन्द्रय आयोजनाहरु संचालनमा आए । सोहि वर्ष जल उत्पन्न प्रकोप नियन्त्रण विभाग अन्तर्गत ७ डिभिजन र ५ सव डिभिजनको स्थापना भयो । बि.सं. २०७२ सालमा सबै जिल्लाहरुमा सेवा सञ्चालन सहज बनाउने हेतुले सिंचाइ विभाग तर्फ सिंचाइ डिभिजन/सव डिभिजनहरुको संख्या ७३ र सिंचाइ व्यवस्थापन डिभिजनको संख्या १३ पुर्याइयो । त्यस्तै जलउत्पन्न प्रकोप नियन्त्रण विभाग अन्तर्गत २४ डिभिजन, २ सव डिभिजन कार्यलयहरु संचालनमा आए । बि.सं. २०७३ सालमा जलउत्पन्न प्रकोप नियन्त्रण विभागको नाम परिवर्तन गरी जलउत्पन्न प्रकोप व्यवस्थापन विभाग राखियो ।

नेपालको संविधान, २०७२ ले परिलक्षित गरे अनुसार जलस्रोत, सिंचाइ तथा जल उत्पन्न प्रकोप व्यवस्थापन सम्वन्धि केन्द्रिय स्तरका ठुला आयोजना तथा परियोजनाहरु, अन्तर प्रदेशिय बहुउपयोगि आयोजनाहरू, अन्तरदेशिय सरोकार रहने बहुउद्देश्यीय आयोजनाहरूको तर्जुमा, नीति निर्माण, कार्यान्वयन तथा अनुगमनमा संघबाट गर्नुपर्ने कार्यहरूको लागि साविकका सिंचाइ विभाग र जलउत्पन्न प्रकोप व्यवस्थापन विभाग खारिज भई बि.सं. २०७४ मा जलस्रोत तथा सिंचाइ विभागको स्थापना भएको हो ।

दुरदृष्टि

1. जलस्रोत तथा सिंचाइ क्षेत्रको समुचित विकास, प्रभावकारी उपयोग तथा दिगो व्यवस्थापनवाट देशको आर्थिक तथा सामाजिक समृद्धि हासिल गर्ने ।

लक्ष्य तथा ध्येय

1. एकिकृत जलश्रोत व्यवस्थापनको माध्यमवाट सन्तुलीत, स्थिर तथा दिगो आर्थिक विकास हाशिल गर्ने ।
2. दिगो एवं भरपर्दो संरचनागत तथा गैर संरचनागत (hardware and software) प्रविधि रजनचेतनाअभिवृद्धिलगायतकाकार्यहरुगरीजलाधारव्यवस्थापनतथाजलउत्पन्नप्रकोपन्युनीकरणएवंव्यवस्थापन बाट आर्थिक सामाजिक हानि नोक्सानी न्यूनीकरण गर्ने ।
3. जलश्रोत तथा सिंचाइ संरचनाहरुको उचित विकास, मर्मत संभार तथा व्यवस्थापन गरी कृषियोग्य भुमिमा बर्षै भरी दिगो एवं भरपर्दो सिंचाइ सुविधा उपलब्ध गराई कृषि उत्पादकत्व तथा उत्पादन वृद्धि गरी गरिवि निवारणमा टेवा पुर्याउने ।

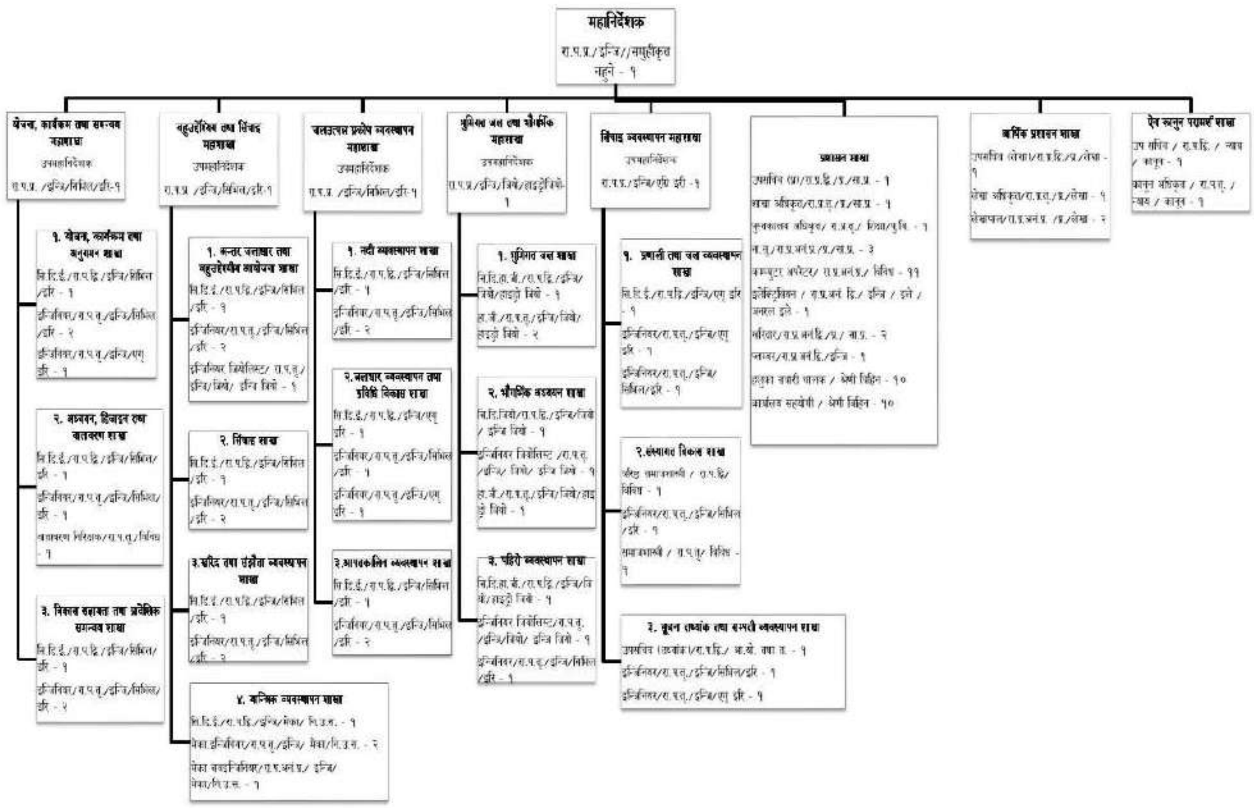
जलस्रोत तथा सिंचाइ विभागको मुख्य उद्देश्यहरू:

- उपलव्ध जलस्रोतको समुचित उपयोग गर्न सिंचाइ गुरुयोजनाको मर्मलाई आत्मसात गर्दै सिंचाइ क्षेत्रको दिगो विकास एवं विस्तार गर्दै कृषि उत्पादकत्वको अभिवृद्धि गर्ने ।
- कृषि योग्य जमिनमा सिंचाइको विकास तथा विस्तार गरी वर्षैभरी भरपर्दो सिंचाइ सुविधा पुऱ्याउने ।
- सिंचाइ तथा जल उत्पन्न प्रकोप व्यवस्थापनका लागि निर्मित संरचनाहरूको समयमै उचित मर्मत संभार गरी दिगो र भरपर्दो बनाउने साथै प्रभावकारी जल व्यवस्थापन तथा आधुनिकीकरण गर्ने ।
- निर्माण संपन्न भइ संचालनमा रहेका ठुला सिंचाइ प्रणालीहरुको दक्षता)Efficiency) बृद्धि तथा उपभोक्ताहरुको क्षमता विकास गरी सिंचाइ सेवालाइ दिगो, भरपर्दो र प्रभावकारी वनाउन सिंचाइ प्रणालीहरुको आंशिक वा पुर्ण रुपमा ब्यवस्थापन हस्तान्तरण गर्दै जाने ।
- नदी तथा जलाधार व्यवस्थापनका लागि उपयुक्त प्रविधिको विकास तथा प्रयोग आवश्यक संरचनागत तथा गैरसंरचनागत कार्य गरी जल उत्पन्न प्रकोपबाट हुने क्षति न्यूनिकरण गर्ने साथै जोखिम रहित जग्गा उकास गर्ने ।
- ठुला प्रकृतिका पहिरोहरुको ब्यवस्थापनका लागि आवश्यक अध्ययन, अनुसन्धान र संरचनागत तथा गैरसंरचनागत कार्य गरी जनधनको क्षति न्यूनिकरण गर्ने ।
- सिंचाइ तथा जलउत्पन्न प्रकोप व्यवस्थापनका पुर्वाधार सम्बन्धी सूचना व्यवस्थापन प्रणालीको विकास तथा सुदृढीकरण गर्ने ।
- सिंचाइ, जल उत्पन्न प्रकोप, पहिरो र जलाधारको विकास एवं व्यवस्थापन को लागि संस्थागत विकास , संगठनात्मक सुधार तथा यस क्षेत्रमा कार्यरत जनशक्तिहरूको क्षमता अभिवृद्धि गर्ने ।
- जलवायु परिवर्तनको प्रभावलाई सम्बोधन गर्न जलवायु जोखिम व्यवस्थापन सम्बन्धी कार्यक्रमहरु अनुकूलन र अल्पीकरण) Adaptation and Mitigation) अवधारणलाई जलश्रोत तथा सिंचाइको विकास कार्यमा समावेश गर्दै सञ्चालन गर्ने ।

उर्जा, जलश्रोत तथा सिंचाई मन्त्रालय

अनुसूचि-४.१.१

जलस्रोत तथा सिंचाइ विभागको दरवन्दी संरचना



जलस्रोतका प्रमुख मुद्दा तथा सवालहरु

- सम्भावना र उपलब्धताको तुलनामा आर्थिक तथा सामाजिक लाभ लिन नसकेको
- जलस्रोतको परिभाषा र गुणस्तरको स्तर तिब्र रुपमा खस्किदै जानु
- जालजन्य प्रकोपहरु दिनानुदिन बढ्दैजानु
- जलस्रोत मात्रै होइन जलाधारहरु समेत तिब्ररुपमा अतिक्रमण हुदै जानु
- सिमा तथा अन्तर्राष्ट्रिय नदी नालाहरुको उपयोग र व्यवस्थापनको क्षेत्रमा छिमेकिदेशहरु संग सन्तुलित एवं प्रभावकारी सम्बन्ध नहुनु
- जलीय जैविक विविधता तथा पर्यावरणीय माथिको संकट उत्पन्न हुनु

- जलस्रोतको उचित व्यवस्थापन तहत संरक्षण एवं नियमन गर्न तथा यस क्षेत्रमा उठने विवाद समाधानको लागि जिम्मेवार निकाय एवं कानूनको अभाव हुनु
- देशको वर्तमान संविधान बमोजिमको संरचनामा जलस्रोत माथिको क्षेत्रधिकार्ले कानुनी रुपमा स्पस्टता हुन् नसक्नु
- जाल्वायु परीवर्त, हिम क्षेत्रको संकुचन अतिवृष्टि र ,हिमतालहरूको विष्फोटन , अनावृष्टि जस्ता प्राकृतिक विपद बाट न्यून असार गर्ने अनुकुलन कार्यक्रम हरु लागू गर्ने
- भूमिगत जलस्रोतको अत्याधिक दोहन बाट जोगाउने कार्यक्रम नहुनु
- जलस्रोतको विकास तहत उपयोग र त्यसमा सामाजिक, आर्थिक, जैविक विविधतामा परेको असर बिषयको एकिकृत तथ्यांक सूचना प्रणाली र अध्ययन नहुनु
- जलवायु परिवर्तन अनुकुल सिंचाई संरचना हरु नहुनु
- जालस्रोतको एकिकृत एवं बहुआयामिक विकास गर्दा तल्लो तटीय क्षेत्र्देश लाइ हुने / लाभ र जोखिम बारे नीतिगत व्यवस्था नहुनु

नेपालमा सिंचाई क्षेत्रको समस्या तथा चुनौतिहरु

हालको अवस्था

देशकोकुल २५लाख ३०हजार हेक्टर सिंचाइ योग्य जमिन रहेकोमा हालसम्म १५लाख ०२हजार हेक्टर जमिनमा सिंचाइ सुविधाका पूर्वाधार विकास भएको छ जसमध्येकरिब एक तिहाइ सिंचित भूमिमा मात्र बर्षैभरी सिंचाई सुविधा उपलब्ध हुने अवस्था रहेको छ । राष्ट्रिय गौरवका रुपान्तरणकारी आयोजनाहरु, ठूला ,बहुउद्देश्यीय तथा अन्तर जलाधार जल पथान्तरण आयोजनाहरु केहि निर्माणको क्रममा रहेका छन् भने केहि निर्माण तयारीको क्रममा रहेका छन। करिव ५० प्रतिशत सिंचित क्षेत्र किसानद्वारा व्यवस्थित सिंचाइ प्रणाली अन्तर्गत पर्दछ । निर्माण सम्पन्न भइसकेका साना आयोजनाहरु उपभोक्ताहरुलाई हस्तान्तरण गर्ने नीति बमोजिम सम्बन्धित उपभोक्ताहरुबाट सञ्चालन भइहेका छन् भने मझौला तथा ठूला सिंचाइ आयोजनाहरू संयुक्त व्यवस्थापनको अवधारणा अनुसार सञ्चालनमा रहेका छन्। सिंचाइ प्रणालीमा क्रियाशील उपभोक्ता संस्थाहरुलाई सुदृढ पार्ने र सिंचाइ विकास कार्यक्रम अझ प्रभावकारी बनाउने अभिप्रायले उपभोक्ताको संस्थागत विकास कार्यलाई सिंचाइ विकास कार्यक्रम अन्तर्गत संस्थागत गरिएको छ ।

समस्याहरु

कृषियोग्य भुमिमा बर्षैभरी आवश्यक सिंचाइ सुविधा उपलब्ध गराउन नसक्नुनिर्माण सम्पन्न भैसकेका , सिंचाइ प्रणालीहरुको नियमित रुपमा सञ्चालन, मर्मत सुधार तथा व्यवस्थापनको लागि Standard Operating Procedure तयार गरि लागु गर्न नसक्नु; सिंचाइ सुविधा पुगेको वा पुग्न सक्ने भुमिमा खण्डिकरण, अव्यवस्थित शहरीकरणका साथै अन्य गैरकृषिजन्य उपयोगलाई रोक्न नसक्नु; अव्यवस्थित एवं असामानरुपमा भैरहेको बसाइसराइलाई रोक्न आबश्यक कानूनी तथा संस्थागत व्यवस्था हुन नसक्नु; सिंचाइ सुबिधा पुगेको भूमिमा बाली विविधिकरण, व्यवसायिकरण तथा यान्त्रिकरणका कार्यक्रमहरु प्रभावकारीरुपमा संचालन हुन नसक्नु; सिंचाइ प्रणालीको दिगो व्यवस्थापनको लागि जल उपभोक्ता संस्थाको क्षमता विकास अपेक्षितरुपमा हुन नसक्नु; हाल सम्म सिंचाई सुबिधा पुग्न नसकेका कृषि योग्य भूमिमा सिंचाई सुबिधा विस्तार गर्न नयाँ, उपयुक्त र सम्भाब्या प्रविधिको छनौट सम्बन्धि डिजाइन मापदण्ड तयार नहुनु, सिंचाइ तथा जल उपयोग दक्षता न्यून हुनु, ठूला तथा बहुउद्देश्यीय आयोजनामा पर्याप्त लगानी हुन नसक्नु आदि समस्याका रुपमा रहेका छन् ।

चुनौतिहरु

सतह सिंचाइ र भुमिगत सिंचाइको संयोजनात्मक उपयोग मार्फत् कृषियोग्य भुमिमा बर्षैभरी सिंचाइ सुविधा उपलब्ध गराउनु; सिंचाइ आयोजनाहरु तोकिएको समय, लागत र गुणस्तरमा सम्पन्न गर्नु; नदी बेसिन योजना र सिंचाइ गुरुयोजनालाई कार्यान्वयनमा ल्याई एकिकृत जलस्रोत व्यवस्थापनको अवधारणा अनुरुप जलस्रोतको बहुउपयोगमा जोड दिंदै सिंचाइ विकास तथा विस्तार गर्नु; ठूला, बहुउद्देश्यीय तथा अन्तरजलाधार जल पथान्तरण परियोजनाहरु कार्यान्वयन गर्न सक्ने संस्थागत व्यवस्था एवं दक्ष प्राविधिक जनशक्ति तयार गर्नु; सिंचाइ पूर्वाधार विकास तथा व्यवस्थापनका लागि संघ, प्रदेश र स्थानीय तह बिच उपयुक्त जिम्मेवारी तथा कार्यक्षेत्र बांडफांट गरी लगानीको प्राथमिकता निर्धारण गर्दै समन्वयात्मक एवं सामन्जस्ययुक्त रुपमा कार्य गर्नु; जलवायु परिवर्तनका कारण पानीको उपलब्धतामा परिरहेको प्रतिकुल असरलाइ न्यूनिकरण गर्नु ; प्राकृतिक नदी प्रणालीमा आधारित रहेर निर्माण गरिएका अधिकांश सिंचाइ प्रणालीबाट बर्षैभरी एकैनासले सिंचाइ सुविधा पुर्याउन उपयुक्त वैकल्पिक जलस्रोतको पहिचान गरी कार्यान्वयन गर्नु; संघियता अनुकुलको योजना, ऐन तथा नियमहरु तर्जुमा गर्दै स्रोत उपयोग तथा लाभ लागत बांडफांट सम्बन्धि मापदण्डहरु तयार गर्नु; सिंचित कृषि क्षेत्रमा सिंचाइ तथा जल उपयोग दक्षता अभिवृद्धि गर्नु; ठूला, बहुउद्देश्यीय तथा अन्तरजलाधार जल स्थानान्तरण आयोजनाहरुको संचालन एवं दिगो व्यवस्थापनको लागि मोडालिटी तयार गरी लागु गर्नु सिंचाइ विकास एवं सञ्चालनमा व्यवस्थापन सूचना प्रणालीको प्रभावकारी ; व्यवस्था गर्दै अनुसन्धान एवं विकास)Research & Developmentलाई प्रोत् (साहन गर्नु आदि चुनौतीहरु रहेका छन् ।

जलस्रोतको वर्तमान अवस्था

- जलस्रोतको धनी देशको रुपमा चिनिएको हाम्रो देश नेपालभित्र रहेका नदी नालाहरुको सरदर वार्षिकबहाव)Average Annual Run off (करिब २२५ अर्ब घन मिटर रहेको अनुमान गरिएको छ ।पुनर्भरणयोग्य भूमिगत जल)Rechargeable Ground Waterअर्ब घन ५.११र ८.५ (

अर्ब ८.२८मिटरकोबीचमा रहेको अनुमान गरिएको छ । उपलब्ध जलस्रोतमध्ये सिंचाइमा घन मिटर, उद्योगमा ०मिटर पानी उपयोग अर्ब घन ०१.०अर्ब घन मिटर र सेवा क्षेत्रमा ५. अमेरिकन डल ६.०भइरहेको अनुमानछ । हाल नेपालको जल उपयोग दक्षता र प्रति घन मिटर रहेको छ । यसहिसाबले आर्थिक तथा सामाजिक रुपमा पानीको उपयोग न्यून छ ।

- जलस्रोतको एकीकृत विकासको लागि जलस्रोत ऐन, २०४९ लागू भईरहेको छ भने जलस्रोत रणनीति, २००२ र राष्ट्रिय जल योजना, २००५ समेत कार्यान्वयनमा रहेका छन् ।

सिंचाई तथा जल उत्पन्न प्रकोप व्यवस्थापन

- देशको कुल क्षेत्रफल १,४७,१८,१०० हेक्टर मध्ये कुल कृषि योग्य क्षेत्रफल २६,४१,००० हेक्टररहेकोमा कुल सिंचाइ योग्य जमीन करिव १७,६६,००० हेक्टर मात्र छ । हालसम्म कुल १४,३३,२८७हेक्टर जमीनमा सिंचाइ संरचनाहरु निर्माण भई सिंचाइ सुविधा पुऱ्याइएको छ । कुल सिंचित क्षेत्रफलमध्ये भूमिगत सिंचाइबाट भएको सिंचित क्षेत्रफल ४,४३,३६५हेक्टर, कृषक व्यवस्थित सिंचाइप्रणालीबाट भएको सिंचित क्षेत्रफल १,६७,९२५ हेक्टर तथा सतह सिंचाइबाट सिंचित भएको क्षेत्रफल८,१३,०६७ हेक्टर छ । नेपालको कुल सिंचित क्षेत्रमध्ये करिब एक तिहाई भूभागमा मात्र वर्षै भरीसिंचाइ सुविधा पुगेको छ । –
- अधिकांश नेपाली जनताले अझै पनि कृषिलाई जीविकोपार्जनको मुख्य आधार बनाइरहेको सन्दर्भमासिंचाइ क्षेत्रमा राज्यले गर्नुपर्ने लगानी पर्याप्त हुन सकिरहेको छैन । विगत एक दशकको परिदृश्य हेर्दासिंचाइको बजेट वार्षिक रुपमा केही बढे पनि यसले यथार्थ मागलाई सम्बोधन गर्न सकेको छैन ।
- सिंचाइ क्षेत्रमा राष्ट्रिय गौरवका विभिन्न आयोजनाहरु सञ्चालन हुँदै आएका छन् । बाँके जिल्लाकोराप्ती नदीमा व्यारेज निर्माण गरी ४२,७६६ हेक्टर जमीनमा सिंचाइ सुविधा उपलब्ध गराउने उद्देश्यलेसिक्टा सिंचाइ आयोजना, कैलाली जिल्लाको ३८,३०० हेक्टरमा वर्षैभरी सिंचाइ सुविधा पुऱ्याइ कृषिउत्पादन वृद्धि गर्ने उद्देश्यले रानी जमरा कुलरिया सिंचाइ आयोजना, बर्दिया जिल्लाको ३६,०००हेक्टरमा वर्षैभरी सिंचाइ सुविधा पुऱ्याई कृषि उत्पादन वृद्धि गर्ने उद्देश्यले बबई सिंचाइ आयोजनातथा बाँके र बर्दिया जिल्लाको थप १५,००० हेक्टर गरी कुल ५१,००० हेक्टर जमीनमा वर्षैभरीसिंचाइ सुविधा उपलब्ध गराउने उद्देश्यले भेरी बबई डाइभर्सन बहुउद्देश्यीय आयोजना ४८र प्रतिसेकेण्ड पानी बबई नदीमा खसाली घन मिट ४0भेरी नदीबाट) मेगावाट विद्युत उत्पादन समेत गर्ने गरीनिर्माण चरणमा रहेका छन् ।(
- मोरङ र सुनसरी जिल्लाको करिब ६८,००० हेक्टर जमीनमा सिंचाइ सुविधा उपलव्ध गराउनेउद्देश्यले सुनसरी मोरङ सिंचाइ आयोजना, सर्लाही, रौतहट, बारा, धनुषा, महोत्तरी जिल्लाका क्षमता१,२२,००० हेक्टर जमीनमा सुनकोशी मरिन डाईभर)्सनसहितसिंचाइ सुविधा पु (ऱ्याउने उद्देश्यलेबागमती सिंचाइ आयोजना तथा कैलाली र कञ्चनपुरको ३३,५२० हेक्टर जमीनमा भरपर्दो सिंचाइसुविधा पुऱ्याउने उद्देश्यले महाकाली सिंचाइ आयोजना निर्माणाधीन (तेश्रो चरण), दाङ जिल्लाको५६,००० हेक्टर जमीनमा सिंचाइ पुऱ्याउने उद्देश्यले वृहत् दाङ उपत्यका सिंचाइ

आयोजना र गोरखाजिल्लाको २,००० हेक्टर जमीनमा सिंचाइ पु¥याउने उद्देश्यले पालुङटार कुन्दुटार सिंचाइ आयोजनानिर्माण एवम् विकासको प्रक्रियामा छन् ।

- सिंचाइ नीति अनुरुप तराईमा १०० देखि २,००० हेक्टरसम्म र पहाडमा १० देखि ५०० हेक्टरसम्मकासिंचाइ प्रणालीहरु विकास गर्ने उद्देश्यले योजनाहरुको निर्माण र पुनरुथापना गर्ने गरी आ२०६१.व.⁄६२ देखि मझौला सिंचाइ कार्यक्रम सञ्चालनमा रहेको छ । यो कार्यक्रमबाट हालसम्म ४४५योजनाहरु सम्पन्न भई ५८,४०३ हेक्टर जमीनमा व्यवस्थित र भरपर्दो सिंचाइ सेवा उपलब्धगराइएको छ ।
- ससाना मूलको पानीको उपयोग –, वर्षाको पानी सञ्चय तथा खोलाको पानीलाई लिफ्ट सोलार) गरी सिमान्तकृत (रविद्युतीय, पिछडिएका कृषकहरुका ससाना पाखा तथा टारलाई समेत – तरकारी) समावेश गरीनगदे बाली, फलफुल, जडिबुटीमार्फत आय आर्जन अभिवृद्धि गर्ने (लक्ष्यका साथ सञ्चालितनयाँ प्रविधिमा आधारित सिंचाइ आयोजनाबाट हाल सम्म ४३६ योजनाहरु सम्पन्न गरी ५,८१० हेक्टरजमीनमा सिंचाइ सुविधा उपलब्ध भएको छ ।द्दढ। समृद्धतराईजिल्ला २२मधेश सिंचाइ विशेष कार्यक्रमबाट –, कर्णाली अञ्चल सिंचाइ कार्यक्रमबाट ५जिल्ला तथा सेती महाकाली सिंचाइ कार्यक्रमबाट ९ जिल्लामा सिंचाइ सम्बन्धी विशेष कार्यक्रमसञ्चालनमा छन् ।
- भूमिगत जलभण्डारको उपयोग गरी तराई तथा भित्री मधेशमा वर्षैभरी सिंचाइ सेवा उपलब्ध गराउनेउद्देश्यलेस्यालो तथा डिप ट्युववेल सिंचाइ आयोजना कार्यान्वयनमा रहेको छ । आ .व. ०७३⁄७४सम्म भूमिगत सिंचाइतर्फ स्यालो टयूववेल र डिप टयूववेल निर्माणबाट करिव ४,१५,६५३ हेक्टरकृषियोग्य भूमिमा सिंचाइ सुविधा उपलब्ध भएको छ ।
- पहाडी भेगका २२ जिल्लाका नदी किनारमा उपलव्ध टार, फाँट तथा खेतहरुमा पनि भूमिगत जलसिंचाइको कार्यक्रम सञ्चालनमा रहेको छ ।घद्द। वैदेशिक सहायतामा सञ्चालित कार्यक्रम अन्तर्गत नेपालको पश्चिमी क्षेत्रका ४२ जिल्लामासञ्चालित सिंचाइ तथा जलस्रोत व्यवस्थापन आयोजना)IWRMPजिल्लाका ३५ क्षेत्रका चालू आर्थिक वर्षमा सम्पन्नहुँदैछ । पूर्व र मध्य (कृषक व्यवस्थित सिंचाइ योजनाहरुको पुनरुथापना गरीभरपर्दो सिंचाइसेवाउपलव्ध गराउने उद्देश्यले सञ्चालित समुदाय व्यवस्थित सिंचित कृषि क्षेत्रआयोजना CMIASP-AFर सिंचाइ पुनरुथापना आयोजना सञ्चालनमा रहेका छन् ।
- निर्माणाधीन तथा सम्पन्न सिंचाइ योजनाहरुमा जल उपभोक्ता समितिहरु क्रियाशील छन् । यस्तासमितिहरु ठूला आयोजनामा करिब ३० र मझौला आयोजनामा करिब ५,००० वटा रहेका छन् ।आगामी दिनमा यी संस्थाहरुको स्रोत परिचालन गर्न सहकारीको अवधारणामा सञ्चालन र क्षमताअभिवृद्धि गर्न आवश्यक रहेको छ ।घद्ध। कमाण्डक्षेत्रमा वर्षै भरी सिंचाइ पु¥याउने उद्देश्यका साथ बहुउद्देश्यीय आयोजनाहरु अन्तर्गतसुनकोशी मरिन डाइभर्सनबहुउद्देश्यीय आयोजनाको विस्तृत् अध्ययन, काली गण्डकी तिनाउडाइभर्सन बहुउद्देश्यीय आयोजना र शारदा दाङ डाइभर्सन बहुउद्देश्यीय आयोजनाहरुको सम्भाव्यताअध्ययन कार्य भइरहेको छ ।
- विभिन्न नदीहरुबाट भइरहेका कटान, डुवानलाई नियन्त्रण र व्यवस्थापन गर्न विभिन्न कार्यक्रमहरुसञ्चालनमारहेका छन् । यसबाट आ२०७४ .व.⁄०७५ सम्म करिब १,०००

कितटबन्ध रसंरचनाहरु .मी., विभिन्न स्थानमा नदी च्यानलाइजेशन, बायोइन्जिनियरिङ्ग, संरचना सबलिकरण, टार रबस्ती संरक्षण तथा पहिरो नियन्त्रणका कामहरु सम्पन्न भएका छन् ।

- बागमती, कमला र लालबकैया नदीहरुमा तटबन्ध निर्माण, सबलिकरणका कामहरु सम्पन्न हुनेक्रममा रहेका छन् । जनसहभागितामा आधारित जनताको तटबन्धको कार्यक्रम, जल उत्पन्न प्रकोपन्यूनीकरण सहयोग कार्यक्रम, रूपा ताल एकीकृत विकास परियोजना तथा आपतकालीन तथा बाढीपहिरो पुनर्निर्माण विशेष कार्यक्रम संचालनमा रहेका छन् ।

समस्या तथा चुनौती

जलस्रोत

- जलस्रोत रणनीति, २००२ र राष्ट्रिय जल योजना, २००५ ले तय गरेका लक्ष्यहरु मध्ये कतिपय लक्ष्यहरु हालसम्म हासिल हुन सकेका छैनन् ।
- नेपालकोशी) भारत बीच भएका जलस्रोत सम्बन्धी द्विपक्षीय सन्धी–, गण्डकी, महाकाली(, सम्झौता तथा सहमति अनुसारका विभिन्न कार्यहरु हुन सकेका छैनन् ।
- एकीकृत राप्ट्रिय जलश्रोत नीतिको खाँचो महसुस भएको लामो समय व्यतित भए तापनि हालसम्म सोको निर्माण हुन सकेको छैन ।
- उपलब्ध जलस्रोतको बाँडफाँड गर्ने सम्बन्धमा संघ, प्रदेश र स्थानीय तह बीच हुन सक्ने विवाद व्यवस्थापन गर्न आवश्यक नीतिगत तथा कानूनी संरचना र क्षमताको विकास गर्न आवश्यक छ ।
- यस क्षेत्रका महत्वपूर्ण दस्तावेज तथा तथ्यांकहरु संकलन, भण्डारण तथा व्यवस्थापन चुस्त दुरुस्त हुन सकेको छैन र निर्णय प्रक्रियामा तिनको प्रयोग गर्ने परम्पराको विकास हुन सकेको छैन ।
- जलस्रोतको एकीकृत एवम् बहुआयामिक विकास गर्दा तल्लो तटीय देश तथा क्षेत्रहरुलाई हुने लाभ र जोखिम बाँडफाँडको नीतिगत व्यवस्थाको अभाव रहेको छ ।

सिंचाइ तथा जल उत्पन्न प्रकोप व्यवस्थापन

अधिकांश सिंचाइ संरचनाहरु रन अफ द रिभर अवधारणामा बनेकाले वर्षातको समयमा बढीभएको पानी संकलन गरी सिंचाइको भरपर्दो सेवा पुऱ्याउन सकिएको छैन ।

- सिंचाइ तथा जल उत्पन्न प्रकोप व्यवस्थापनको क्षेत्रमा लिइएको लक्ष्य हासिल गर्न पर्याप्त रकमविनियोजन हुन सकेको छैन ।
- मझौला सिंचाइ आयोजना, नयाँ प्रविधिमा आधारित सिंचाइआयोजना, डिप ट्युववेल सिंचाइ आयोजना तथा नदीजन्य प्रकोप व्यवस्थापन क्षेत्रमा अत्यधिकमाग रहेको र त्यस्ता आयोजनाहरुमा बहुवर्षीय ठेक्का सम्झौताको कारणले धेरै दायित्व सिर्जनाभएको छ ।
- कृषि विकास रणनीतिले परिदृष्य गरेअनुरुपकार्यक्रम सन्चालन गर्न नसक्नु तथा यस रणनीतिमा उल्लेख भएका कृषि विकाससँग सम्बन्धितनिकायहरु बीच समन्वय नहुनु समस्याको रुपमा रहेका छन् ।

- नेपाल सरकारले विगतमा आफ्नो लगानीमा निर्माण गरेका थुप्रै सिंचाइ आयोजनाहरु जीर्णअवस्थामा छन् ।निर्माण सम्पन्न भई सरकारले व्यवस्थापन गरेका, संयुक्त व्यवस्थापनमा रहेकातथा व्यवस्थापन हस्तान्तरण भएका कार्यक्रमहरु प्रभावकारी हुन सकेका छैनन् ।
- जल उत्पन्न प्रकोप व्यवस्थापनको क्षेत्रमा विनियोजित बजेट क्षति र आवश्यकताको दाँजोमान्यून भएकोले एकीकृत रुपमा प्रभावकारी संरचना निर्माण गर्न सकिएको छैन । निर्मितसंरचनाको मर्मत सम्भारका लागि पर्याप्त बजेट नहुने र स्थानीयहरुको समेत उचित परिचालनहुन नसक्नाले यस क्षेत्रमा अपेक्षित उपलब्धी हासिल हुन सकेको छैन ।
- सिंचाइ सुविधा उपलब्ध भएका एवम् हुने जमीन जथाभावी रुपमा खण्डीकरण गरी अन्यप्रयोजनको लागि प्रयोग गर्न तथा जोखिमयुक्त क्षेत्रको जमीनको उपयोगमा रोक लगाउनसकिएको छैन ।
- जग्गा प्राप्तिमा चलन चल्ती र सरकारी दर भन्दा धेरै मुआब्जा माग हुने गरेको छ ।
- जल उपभोक्ता संस्थाहरु सवल र सक्षम बन्न सकेका छैनन् ।
- सिंचाइ नीतिले परिलक्षित गरेको सिंचित क्षेत्रको घोषणा तथा सहकारी अवधारणामा सिंचाइप्रणालीको विकास हुन सकेको छैन ।
- जल उत्पन्न प्रकोप व्यवस्थापन नीतिले व्यवस्था गरे अनुरुपको आपतकालिन विपद्को संबोधनगर्ने स्रोतको व्यवस्था हुन सकेको छैन ।
- नदी क्षेत्रलाई क्षेत्राङ्कन)Zoningगर्न सकिएको छैन । नदी किनाराहरु अतिक्रमण गरी (बस्तीविकास तथा जग्गा प्लटिङ गर्ने कार्यमा रोक लगाउनसकिएको छैन । यसले गर्दा नदीनियन्त्रणका संरचना भएको छ निर्माणमा विवाद हुन गई निर्माण कार्य प्रभावित (तटबन्ध) ।नियमन गर्ने कानुनको अभावमा यस्ता कार्य गर्नेलाई दण्डित गर्न सकिएको छैन ।
- नेपालभारतबीच सिंचाइ–, बाढी नियन्त्रण तथा डुवानको विषयमा देखिएका समस्याहरुकोसम्बन्धमा द्विदेशीय समझदारी निर्माणका लागि कुटनीतिक पहल गर्न आवश्यक छ ।
- भू) त्रको बढ्दो क्षयीकरणबाट हुने गेग्रान बहावक्षय रोकथाम तथा जलाधार क्षे–Sedimentका (कारण नदी पिंधको वृद्धि नियन्त्रण गर्न तथा उत्पादित नदी जन्य निर्माण समाग्रीकोप्रभावकारी व्यवस्थापन गरी प्रकोप न्यूनीकरण गर्न सकिएको छैन ।
- जलवायु परिवर्तनको कारणबाट पानीको उपलब्धतामा आएको अनिश्चितता, बाढी, खडेरी, पहिरो, भूक्षय जस्ता क्षेत्रमा भएको फेरबदल र त्यसबाट उत्पन्न जोखिम व्यवस्थापन गर्नुकासाथै –) जलवायु परिवर्तन अनुकुलन संरचनाClimate Change Adaptive Infrastructure(को निर्माण गर्नुपर्ने चुनौती रहेको छ ।

भावी मार्गचित्र

नेपाल सरकारकोनेपाल समृद्ध÷, सुखी नेपाली–२०७५अभियानको प्रभावकारी कार्यान्वयन गर्न '८५ को अवधिलाई ऊर्जा तथा जलस्रोत दशक÷को रुपमा अघि बढाइनेछ ।

नितिगत मार्गचित्र

जलस्रोत

- नेपालको संविधान बमोजिम सबै तहका सरकार तथा उपयोग कर्ताहरूबाट विवादरहित ढंगले जलस्रोतको विकास, संरक्षण, व्यवस्थापन, नियमन र एकीकृत, समन्वयात्मक एवम् बहुआयामिक उपयोगबाट अधितकम लाभ हासिल गर्नका लागि एकीकृत राष्ट्रिय जलस्रोत नीति निर्माण गरी कार्यान्वयनका लागि कानूनी तथा संरचनागत व्यवस्था गरिनेछ । यस अन्तर्गत जल तथा उर्जा आयोगको वेसिन कार्यालयहरु स्थापना समेतबाट आयोगको पुनर्संरचना गरी थप प्रभावकारी बनाइनेछ ।
- एकीकृत जलस्रोत व्यवस्थापनको सिद्धान्त अनुरुप जलस्रोतको बहुआयामिक उपयोग र अन्तर जलाधार जल स्थानान्तरण Inter Water Basin Transfer को सम्भावना समेतलाई विचार गरी उपलब्ध जलस्रोतको आर्थिक, सामाजिक एवम् वातावरणीय क्षेत्रमा अधिकतम लाभ हुने गरी सबै नदीको वेसिन योजना र सो बमोजिम विभिन्न उपयोगगत गुरुयोजना समेत आगामी तीन वर्षभित्र तयार गरी जलस्रोतको योजनावद्ध विकास, व्यवस्थापन र उपयोग गरिनेछ ।
- भूमिगत जलस्रोतको विकास, व्यवस्थापन, संरक्षण, नियमन र गुणस्तर कायम गर्न जलस्रोत ऐनमा आवश्यक व्यवस्था गरिनेछ ।
- संघीय संरचनामा संघप्रदेश –, प्रदेशप्रदेश र स्थानीय तहबीच उपलब्ध जलस्रोतको अधिकतम –) उपयोगOptimum Use तथा व्यवस्थापन सम्बन्धमा भविष्यमा हुनसक्ने विवाद व्यवस्थापन गर्न आवश्यक संरचनागत व्यवस्था गरिनेछ । साथै यस सम्बन्धी कार्यरत जनशक्तिको क्षमता अभिवृद्धि गरिनेछ ।

बहुउद्देश्यीय तथा अन्तर जलाधार स्थानान्तरण आयोजना

- नेपालमा उपलब्ध जलस्रोतलाई बहुआयामिक तथा बहुउद्देश्यीय आयोजनाको रुपमा विकास गरिने छ । यसबाट सिंचित क्षेत्रमा वर्षैभरी पानी उपलब्ध हुने, जलविद्युत, बाढी नियन्त्रण, खानेपानी, जल यातायात, आमोदप्रमो –द, पर्यटन तथा वातावरणीय फाइदा हुनुको साथै उपलब्ध जलस्रोतको अधिकतम उपयोग हुनेछ । तराईमधेसमा भूमिगत जलस्रोतको उपयोगबाट – भूमिगत जल सतह घट्न थालेकोले यस्ता आयोजनाबाट भूमिगत जलस्रोत पुनर्भरणहुनुको साथै मधेस क्षे –क उपयोगबाट तराईसतह र भूमिगत जलस्रोतको संयोजनात्मत्रको कृषि उत्पादकत्वमा वृद्धि हुने छ ।
- अन्तर जलाधार जल स्थानान्तरणका आयोजनाहरुका सम्बन्धमा नीतिगत व्यवस्था गरी कार्यान्वयन गरिनेछ
- प्रादेशिक सन्तुलन, माग र आपूर्ति, उत्पादन सम्मिश्रण Generation Mix तथा ऊर्जा सुरक्षा आदिलाई मध्यनजर गर्दै सम्भव भएसम्म हरेक प्रदेशमा कम्तिमा एउटा जलाशययुक्त आयोजना निर्माण गरिनेछ ।
- अन्तर जलाधार स्थानान्तरण, ठूला तथा जलाशययुक्त र बहुउद्देश्यीय आयोजनाहरुको विकास गर्न सरकारी लगानीले मात्र सम्भव नहुने भएकोले सहुलियतपूर्ण ऋण तथा साझेदारी लगायतका वैकल्पिक लगानीको मोडल विकास गरी कार्यान्वयन गरिनेछ ।

- बहुउद्देश्यीय प्रकृतिका आयोजनाहरुको बाँध, विद्युतगृह तथा अन्य संरचनाहरुको निर्माण, मर्मत, सम्भार तथा सञ्चालन कार्यको लागि उपयुक्त मोडालिटिको तर्जुमा गरिनेछ ।
- जलाशययुक्त आयोजनाहरुको विकास हुने क्षेत्रमा थप सडक विस्तार एवम् बस्ती विकास लगायत पूर्वाधार निर्माण कामको नियमन गर्न नीतिगत व्यवस्था गरिनेछ ।
- जलाशययुक्त आयोजनाहरु विकास गर्दा तल्लो तटीय देशहरुलाई हुने फाईदाको बाँडफाँड गर्ने मोडालिटी तय गरिनेछ । ज्ञद्द। नेपालभारतबीचका जलस्रोत सम्बन्धी मुद्दाहरुलाई दुई देशको – समझदारी र समन्वयमा नेपालको अधिकतम हित हुनेगरी संबोधन गरिनेछ । यसको लागि नेपालभारतबीचका विभिन्न द्विपक्षीय संयन्त्रहरुलाई प्रभावकारी बनाईनेछ । –

सिंचाइ तथा जल उत्पन्न प्रकोप व्यवस्थापन

- क्षेत्रको अभिवृद्धि सिंचित कृषि÷, संघीय नेपालको समृद्धि भन्ने मूल नाराका साथ सिंचाइ क्षेत्रको ’ विकास गरिनेछ । कृषिका लागि सिंचित क्षेत्र विस्तार गरी वर्षै भरी भरपर्दो सिंचाइ सुविधा पुऱ्याउन अन्तर जलाधार जलस्थान्तरण तथा बहुउद्देश्यीय जलाशययुक्त आयोजनाहरुको कार्यान्वयन गर्नुका साथै भूमिगत सिंचाइको संयोजनात्मक उपयोग तथा नयाँ प्रविधिको प्रयोग गरिनेछ ।
- ÷ नदी किनाराका गरा, सदा हराभराभन्ने नाराका साथ पहाडी क्षेत्रका नदी छेउ छाउँमा ’ अवस्थित कृषि योग्य जमीनमा सौर्य उर्जाको समेत उपयोग गरी लिफ्ट सिंचाइ आयोजनालाई लन गरिनेछ । यसले सीमान्तकृत कृषिभूमि तथा सीमान्तकृत अभियानकै रुपमाम सञ्चा कृषकलाई सम्बोधन गर्दै पहाडी क्षेत्रमा कृषि उत्पादन बृद्धि भई त्यस क्षेत्रको आर्थिक क्रियाकलापमा उल्लेखनीय बृद्धि हुनेछ ।
- तराईमधेसमा उपलब्ध भूमिगत जलस्रोतको अधिकतम उपयोग गर्न स्यालो तथा डिप – निर्माणमा तीब्रता दिइनेछ । निर्माण भएका ट्युववेलहरुमा विद्युत सेवा निरन्तर ट्युववेलको प्रवाह र सोलार प्रविधिको उपयोग गर्न भरपर्दो वितरण लाइनको विस्तार गरिनेछ । दिगो सञ्चालन गर्न नेट मिटरिङ अवधारणाबाट धेरै भएको विजुली ग्रिडमा जोड्ने व्यवस्था मिलाइनेछ ।
- सिंचाइ आयोजनाहरुको प्रभावकारी सञ्चालन र मर्मत सम्भार गर्न उपभोक्ता संस्थासँग संयुक्त व्यवस्थापन, व्यवस्थापन हस्तान्तरण, सार्वजनिक निजी साझेदारी जस्ता उपायहरु अवलम्बन गरिनेछ । सिंचित क्षेत्रमा कृषिका कार्यक्रमको अलावा प्रविधि, मल, बिउ, बजार व्यवस्थापन, सहकारी विकास जस्ता कार्यक्रमहरुको समेत प्रभावकारी कार्यान्वयन गरिनेछ ।
- भूमिगत सिंचाइको लागि डिप ट्युववेल सञ्चालनको लागि आवश्यक कानूनी तथा नीतिगत सुधार गरिनेछ ।
- सिंचाइ सेवा शुल्कलाई व्यवस्थित गर्न नीतिगत तथा कानूनी व्यवस्था मिलाइनेछ ।
- सिंचाइ, जलस्रोत तथा जल उत्पन्न प्रकोप व्यवस्थापनका लागि उपभोक्ता समितिलाई सहकारी अवधारणामा विकास र व्यवस्थापन गर्न नीतिगत तथा कानूनी सुधार गरिनेछ ।
- नदी किनाराहरु अतिक्रमण गरी बस्ती विकास, घर निर्माण र अन्य उपयोग गर्ने कार्यलाई समेत रोक्र ने गरी जलस्रोत ऐनमा आवश्यक संशोधन गरी लागू गरिनेछ । द्धद्द। संघ, प्रदेशस्थानीय

तहका सरकारबीच समन्वय गरी जलस्रोत तथा सिंचाइको विकास र जल उत्पन्न प्रकोप व्यवस्थापनको लागि तीनवटै तहको क्षमता अभिवृद्धि गरिनेछ । प्रदेश तथा स्थानीय तहबाट सिंचाइ विकास गर्न विभिन्न कार्यविधि, नियमावली, निर्देशिकाको नमूना तयार गरिनेछ । जल उत्पन्न प्रकोप व्यवस्थापन गर्न विद्यमान जल उत्पन्न प्रकोप व्यवस्थापन नीतिलाई संघीय संरचना बमोजिम प्रदेश र स्थानीय तहको समेत प्रकोप व्यवस्थापनमा महत्वपूर्ण भूमिका हुने गरी परिमार्जन गरिनेछ ।

कार्यगत मार्गचित्र

जलस्रोत

- निर्माणाधीन भेरीहुउद्देश्यीय आयोजनाको सुरुङबबई डाईभर्सन ब–, बाँध र विद्युतगृह सहितको संरचनाहरुको निर्माण कार्य आर्थिक वर्ष २०७८/७९ भित्र सम्पन्न गरिनेछ ।
- बुढीगण्डकी जलाशययुक्त आयोजना निर्माणका लागि आगामी आर्थिक वर्षभित्र बोलपत्र प्रक्रिया शुरु गरिनेछ । जग्गा अधिग्रहण, पुनर्वास तथा पुनर्स्थापना सम्बन्धी कार्यलाई दुरत गतिमा अगाडि बढाईनेछ ।
- सुनकोशीमरिन डाइभर–कमला डाइभर्सन तथा सुनकोशी–्सन बहुउद्देश्यीय आयोजनाहरुको तुलनात्मक अध्ययन एवम् विश्लेषण गरी उपयुक्त आयोजनाको आगामी आर्थिक वर्षबाट कार्यान्वयन प्रक्रिया अघि बढाइने छ ।
- बागमती नदी जलाधार सुधार आयोजना अन्तर्गतको निर्माणाधीन धाप बाँध आगामी वर्ष सम्पन्न गरी नागमती बाँध निर्माणको प्रक्रिया शुरु गरिनेछ । यसबाट बागमती नदीमा थप ४४० लिटर प्रतिसेकेन्ड जल प्रवाह हुनेछ ।
- २४५ मेगावाट क्षमताको नौमुरे राप्ती बहुउद्देश्यीय आयोजनाबाट हाल निर्माणाधीन सिक्टा) ४३,००० हे(., प्रगन्ना ५),८०० हे(., बड्कापथ ४),००० हेसिंचाइ आयोजनाहरुको कुल (. ५२,८०० हेक्टर कमाण्ड क्षेत्रमा बाह्रै महिना सिंचाइ सुविधा उपलब्ध गराउने साथै कपिलवस्तुको ३१,००० हेक्टरमा वर्षैभरी सिंचाइ सुविधा, बाढी नियन्त्रण, जलविद्युत तथा अन्य फाईदा हुने गरी आगामी आर्थिक वर्षदेखि आयोजना कार्यान्वयन प्रक्रिया अगाडि बढाइनेछ ।
- कालिगण्डकी तिनाउ डाइभर्सन–बहुउद्देश्यीय आयोजनाको विस्तृत् अध्ययन सम्पन्न गरी कार्यान्वयनको प्रक्रिया अगाडि बढाइने छ ।
- मोरङ तथा झापामा सिंचाइ सुविधा विस्तार गर्न कन्काई जलाशययुक्त बहुउद्देश्यीय आयोजना वा तमोरमोरङ स्टोरेज तथा डाइभर्सन बहुउद्देश्यीय आयोजनाको तुलनात्मक विश्लेषण गरी उप–युक्त आयोजनालाई अगाडि बढाईनेछ ।
- क्षेत्रीय विद्युत बजार, बहुपक्षीय लगानी एवम् लाभको प्रवद्र्धन हुने गरी क्षेत्रीय महत्वका ठूला जलाशययुक्त आयोजनाहरु Mega Reservoir Project विकास गरिनेछ । यसका लागि आगामी पाँच वर्षभित्र कर्णाली बहुउद्देश्यीय जलाशययु (चिसापानी)क्त आयोजना १०),८०० मेगावाट (को विस्तृत अध्ययन एवम् वित्तीय व्यवस्था र विकास गर्ने मोडालिटी तयार गरी निर्माण प्रक्रिया अघि बढाइनेछ ।

- आगामी आर्थिक वर्षमा कोशी, गण्डकी तथा कर्णाली नदी हुँदै समुद्रसम्मको जल मार्गको सम्भाव्यता अध्ययन गरिनेछ ।

सिंचाइ तथा जल उत्पन्न प्रकोप व्यवस्थापन

- ठूला सिंचाइ आयोजनाहरुको विकास अन्तर्गत महाकाली सिंचाइ आयोजना (तेश्रो चरण), राष्ट्रिय गौरवका रानी जमरा कुलरिया, बबई तथा सिक्टा सिंचाइ आयोजनाहरु, वाग्मती सिंचाइ आयोजना र सुनसरीमोरङ सिंचाइ आयोजनाहरु तोकिएको समयमा सम्पन्न गर्न सबै स्रोत साधनको – अधिकतम परिचालन गरिनेछ ।
- संघीय सरकारको कार्यक्रमको रुपमा एकीकृत उर्जा तथा सिंचाइ विशेष कार्यक्रम अन्तर्गत आगामी आर्थिक वर्षबाट उर्जा तथा कृषि क्षेत्रसँग समन्वय गरी एकीकृत रुपमा आयोजनाहरु संचालन गरिनेछ । यस कार्यक्रम अन्तर्गत नेपाल सरकारको लगानीमा मध्य पहाडी क्षेत्रको करिब १०,००० हेक्टर कमाण्ड क्षेत्रमा लिफ्ट प्रविधि मार्फत सिंचाइ सुविधा उपलब्ध गराउने उद्देश्यले मध्यपहाडी टार लिफ्ट सिंचाइ आयोजना शुरु गरिनेछ । तराइमधेसका करिब – २२,००० हेक्टर कमाण्ड क्षेत्रमा सौर्य उर्जामा आधारित भूमिगत जलस्रोतको विकासको लागि तराई मधेश सौर्य लिफ्ट सिंचाइ आयोजना अभियानको रुपमा कार्यान्वयन गरिनेछ । –
- यान्त्रिक नवीन सिंचाइ आयोजना)Mechanized Irrigation Innovation Projectअन्तर्गत (तनहु, लमजुङ्ग, पाल्पा, र स्याङ्गजाको १४०० हेक्टर जमीनमा तथा सर्लाही र रौतहटको ४०,००० हेक्टर जमीनमा सौर्य उर्जा जडित लिफ्ट प्रविधिमार्फत सिंचाइ सुविधा उपलब्ध गराउने उद्देश्यले विस्तृत् अध्ययन कार्य सम्पन्न गरी निर्माण कार्य शुरु गरिनेछ ।
- ठूला नदीहरुबाट हुने कटान, डुवानबाट क्षति पुग्ने बस्ती, टार, कृषि भूमि र संरचनाको बचावट गर्न जनसहभागितामा आधारित जनताको तटबन्ध कार्यक्रम, राष्ट्रपति चुरे तराईमधेस नदी – नियन्त्रणका कार्यहरु, राष्ट्रिय पुनर्निर्माण प्राधिकरणका नदी नियन्त्रण तथा पहिरो व्यवस्थापनका कार्यक्रमहरु, महाकाली नदी नियन्त्रण आयोजना (दार्चुला), कर्णाली नदी नियन्त्रण आयोजना, नारायणी नदी नियन्त्रण आयोजना, बबईऔरही नदी व्यवस्थापन आयोजना जस्ता –भादा– कार्यक्रमहरुलाई थप प्रभावकारी बनाइनेछ ।
- तराई मधेसका पश्चिम राप्ती, मावा रतुवा, लखनदेही, मोहना खुटीया, पूर्वी राप्ती तथा वक्राहा नदीहरुमा संरचनागत तथा गैर संरचनागत माध्यमबाट बाढी व्यवस्थापन गर्ने उद्देश्यले प्राथमिकता प्राप्त नदी बाढी जोखिम व्यवस्थापन आयोजना)Priority River Flood Risk Management Projectकार्यान्वयनमा ल्याईनेछ । ज्ञण्द। (
- भौगोलिक स्थितिका कारण चुरे, महाभारत र हिमश्रृखलाहरुमा गएका ठूला पहिरोबाट, बस्ती, जमीन एवम् संरचना बचावट गर्न ठूला पहिरो नियन्त्रण तथा व्यवस्थापन लगायत जलाधार व्यवस्थापनका कार्यक्रमहरु जोखिमयुक्त पहिरो नियन्त्रण र व्यवस्थापन आयोजना अन्तर्गत सञ्चालन गरिनेछ ।
- न र उत्पादकत्वकृषि उत्पाद÷ वृद्धिको लागि सिंचाइको विकासभन्ने नाराको साथ सिंचाइ ', कृषि तथा सहकारीको समन्वयात्मक विकासमा जोड दिइनेछ । यसका लागि हाल सञ्चालित समुदाय

व्यवस्थित सिंचित कृषिक्षेत्र आयोजना)CMIASP-AFलाई प्रभावकारी बनाइनेछ । साथै (, Innovative and Climate Resilient Irrigated Agriculture Project समेत शुरु गरिनेछ ।

- सिंचाइ योजनाहरुको मर्मत सम्भार तथा दिगो व्यवस्थापनमा जोड दिईनुका साथै ठूला एवम् बृहत सिंचाइ योजनाहरुको पुनरुत्थापना कार्यलाई प्राथमिकताका साथ अघि बढाइनेछ ।
- जलवायु परिवर्तनबाट जलस्रोत, सिंचाइ तथा जल उत्पन्न प्रकोप व्यवस्थापनको क्षेत्रमा पर्ने असरको विषयमा अध्ययन, अनुसन्धानलाई थप प्रभावकारी बनाइनेछ । जलवायु परिवर्तनको कारणले विद्यमान संरचना, तापमान बृद्धिको कारणले बालीमा पर्ने असर, भूमिगत जलस्रोतको पानीको सतहमा आउने परिवर्तन, बाढी, पहिरो, सेडिमेन्ट यिल्ड)Sediment Yieldजस्ता विषयमा (अनुसन्धान तथा खोजमुलक कार्यहरु गरिनेछ ।
- माथि उल्लेखित सिंचाइ सम्बन्धी कार्यक्रमहरुबाट आगामी पाँच वर्षमा थप ३,००,००० हेक्टर जमीनमा व्यवस्थित सिंचाइ सेवाका लागि पूर्वाधार विकास भई कूल सिंचित क्षेत्रको करिब ४५ प्रतिशत जमीनमा वर्षैभरी सिंचाइ सेवा उपलब्ध हुनेछ । यसबाट कृषि उत्पादकत्व तथा उत्पादनमा उल्लेखनीय वृद्धि भई आर्थिक समृद्धिमाम टेवा पुग्नेछ । यसै गरी नदी नियन्त्रण सम्बन्धी कार्यक्रमहरुबाट आगामी पाँच वर्षमा ३०० किलोमिटर तटबन्ध तथा नदी नियन्त्रण संरचनाहरु निर्माण भई करिब १,१०० हेक्टर जमीनको उकास गरी उत्पादन मुलक कार्यमा उपयोग गरिनेछ ।

सुशासन, संस्थागत सुधार तथा क्षमता अभिवृद्धि

- उर्जा, जलस्रोत र सिंचाइ क्षेत्रमा गरिने कार्य तथा सेवा प्रवाहमा सुशासन र पारदर्शीतालाई प्रभावकारी रुपमा लागू गरिनेछ ।
- यस क्षेत्रमा नतिजामूलक कार्यव्यवस्थापन तथा जनताप्रतिको उत्तरदायित्व निर्वाह गर्ने उद्देश्यका साथ कार्यक्रम सञ्चालन गरिनेछ ।
- ठेक्का सम्झौता गरी तोकेको समयभित्र निर्माण सम्पन्न नगर्ने निर्माण व्यवसायी तथा अध्ययन सम्पन्न नगर्ने परामर्शदाता कम्पनीहरुसँगको ठेक्का सम्झौता रद्द गरी कालो सूचीमा सुचीकृत गर्ने कार्यलाई प्रभावकारी बनाईनेछ । साथै निर्माण कम्पनीले आफ्नो क्षमता भन्दा बढी काम लिने प्रवृत्तिलाई निरुत्साहित गरिनेछ ।
- आयोजनाहरुलाई समयमा सम्पन्न गरी अधिकतम प्रतिफल हासिल गर्न प्रभावकारी योजना तयारी, आवश्यक बजेट विनियोजन, कर्मचारी व्यवस्थापन, कार्य सम्पादनमा आधारित प्रोत्साहन प्रणाली, प्राविधिक क्षमता अभिवृद्धि तथा उपयुक्त अनुगमन संयन्त्रको विकास गरिनेछ ।
- जलस्रोत क्षेत्रमा अनुसन्धान तथा विकासका साथै कार्यरत जनशक्तिको क्षमता अभिवृद्धि गर्न हालको जलस्रोत विकास तथा अनुसन्धान केन्द्रलाई थप जिम्मेवारीका साथ पुनर्संरचना गरिनेछ । यसबाट जलस्रोतको उपलव्धता र व्यवस्थापन, गेग्रान, जल उत्पन्न प्रकोप, जलवायु परिवर्तन जस्ता क्षेत्रमा अध्ययन, अनुसन्धान र क्षमता अभिवृद्धि सम्बन्धी कार्यहरु प्रभावकारी ढंगले सञ्चालन हुनेछन् ।

- आयोजना कार्यान्वयन तथा अनुगमनका साथैDecision Making Support System लाई छरितो तथा चुस्त हुने गरी तथ्याङ्क व्यवस्थापनलाई प्रभावकारी बनाउन Smart Management, E-governance and E-management जस्ता विधि अवलम्बन गरिनेछ । तथ्याङ्क व्यवस्थापन गर्न डाटा डिजिटाइजेसन, लाइब्रेरी व्यवस्थापन, डाटा युनिट स्थापना गर्ने जस्ता कार्यहरु गरिनेछ ।
- पूर्व सूचना लगायत बाढी व्यवस्थापनका कार्यहरुमा यस क्षेत्रमा कार्यरत विभिन्न गैरसरकारी संस्थाहरुसँग समेत सहकार्य गरी सूचना सहज रुपमा एकलद्वारबाट प्राप्त हुने संरचनाको विकास र उक्त सूचनाको व्यवस्थापन वैज्ञानिक ढंगबाट गरिनेछ । यसको लागि आवश्यक तालिम तथा क्षमता अभिवृद्धिको कार्यक्रम समेत सञ्चालन गरिनेछ ।
- उर्जा, जलस्रोत,
- सिंचाइ तथा जल उत्पन्न प्रकोप व्यवस्थापन सम्बन्धी आयोजनाहरुको प्रभावकारी कार्यान्वयन, व्यवस्थापन तथा ठेक्का व्यवस्थापन गर्न प्राविधिक जनशक्तिहरुलाई तालिम र विभिन्न क्षमता अभिवृद्धिका कार्यक्रम सञ्चालन गरिनेछ ।
- भौतिक पूर्वाधारहरुको निर्माणको कामलाई छिटो छरितो रुपले सम्पन्न गर्न सरकारी निकाय र निजी लगानीकर्ताहरुको समेत लगानीमा साधन स्रोत सम्पन्न निर्माण कम्पनीहरु स्थापना गरिनेछ । यसले गर्दा देश निर्माणमा आत्मनिर्भर भई निर्माण कार्य छिटो सम्पन्न हुनुका साथै रोजगारीको अवसरमा समेत वृद्धि हुनेछ ।

1.7 Overview of Irrigation development in the SAARC region

South Asia, comprising of Bangladesh, Bhutan, India, Nepal, Maldives, Pakistan, and Sri Lanka, is one of the most densely populated and poorest regions of the world. Endemic poverty aects one-third of the population and the region faces signi-cant spatial and periodic water shortages due to the uneven temporal and geographic distribution of rainfall1. Water shortages are set to intensify as the region's population is projected to increase signi-cantly over the next 50 years and climate change adds further uncertainty. Meanwhile, the region boasts one of the highest rates of irrigated agriculture in the world, estimated at around 40% of total cultivated area. The region hosts some of the oldest and largest irrigation systems on earth (Grand Anicut and Cauvery, Indus Basin and Bhakra Nangal canal), however, the area irrigated by these schemes has been stagnant over the last decade and is even on decline since 1990 due to poor operation and maintenance. Water tanks2 in India, Kareze3 in Pakistan and Kuhls4 in the Himalayas have been decreasing in both size and numbers.The exception to the trend is Sri Lanka, where smaller systems and a humid environment help canal irrigation function productively (IWMI and FAO 2009). Increasingly, farmers are opting for private tubewells, which are easier to maintain and operate, and more exible than canal irrigation schemes. This has translated into a groundwater boom in much of South

Asia, most notably in India. Groundwater use for irrigation has become so extensive that experts and governments are now worried about overexploitation and a resulting reduction in future water resources, as extraction rates are exceeding recharge rates. (GWP and IWMI 2011) 1.2 Fresh Water Resources The endowment of freshwater resources in the South Asian region is very varied: the annual precipitation, estimated to be about 1083 mm in India, 2666 mm in Bangladesh, 280 mm in Pakistan, 1500 mm in Nepal, 1712 mm in Sri Lanka and 2091 mm in Myanmar, is accompanied by high temporal and spatial variability resulting in an excess of surface water during the summer months and water shortfalls during the winters, due to which groundwater and surface storage and irrigation systems are of utmost importance for agriculture in South Asian countries. South Asia's primary freshwater resources are presented in Table 1. All the major river basins cross over national boundaries. The Ganges-Brahmaputra-Meghna basin is the largest (and is in fact, the second largest fresh water basin in the world after the Amazon basin). Water from this basin supports 40% of theregion's population. It is followed by the Indus basin which, with less than 1,700 m^3 per capita water availabil- ity, is classified as a water stressed basin and will likely be reclassified as water scarce (below 1000 m^3 per capita) by 2015. The Helmand basin and the much smaller Karnaphuli basin are next.

Table 1 **Freshwater resources of South Asia**

River Basins	Basin Area (km^2)	Per Capita Water Availability (m^3)	Country's Coverage (km^2)	Annual Available Water (billion m^3)
Ganges Brahmaputra[a] Meghna	1,745,000	3,473	Nepal (140,000) India (1,105,000) Bangladesh (129,000) Bhutan (45,000) China (326,000)	2,025
Indus[a]	1,170,838	1,329	Pakistan (632,954) India (374,887) China (86,432) Afghanistan (76,542) Nepal (23)	287
Helmand[b]	306,493	2,589	Afghanistan (262,341) Iran (33,111) Pakistan (11,041)	18
Karnaphuli[b]	12,510	-	Bangladesh (7,400)	-

			India (5,100) Myanmar (10)	

Source: Aquastat 2007[a] and UNEP 2008[b]

Irrigation systems differ across country and region: there are large contiguous centrally controlled irrigation systems in India and Pakistan, and medium-sized ones in Sri Lanka, Bangladesh & Nepal; there are also isolated scale farmers' managed systems in Pakistan, India, Nepal and Bangladesh.

In terms of total renewable water resources, India comes first with an availability of 1911 Km^3, and Bhutan, Sri Lanka and Maldives last with 78.0, 52.8 and 0.03 km^3; (FAO 2011). However, renewable water per capita exhibits quite a different pattern, with Pakistan and the Maldives falling in the water-stressed category and Bhutan coming out on top with 109244m^3/capita, falling in the water-surplus category. With populations set to increase, per capita availability of renewable water is expected to further decrease across the region (FAO2011).

Challenges of Managing Water in South Asia

With a share of 95% of total consumption against a world average of 70% (UNEP 2008), agriculture is by far the highest water-consuming sector in South Asia. Therefore this report focuses predominantly on water use in the agricultural sector. Any policy on water will not be complete until it caters for efficiency calculations across sectors and takes into account allocation decisions on the margin. However, it was felt that agriculture's share in water consumption in the region is so overwhelmingly dominant that focusing our attention sharply on irrigation and water use on farms is more appropriate than providing a cursory overview of water use in non-farm sectors, such as in industries and in settlements. As South Asia industrializes and continues to urbanize, and this share falls, the current choice of focus will need to be reconsidered, but agriculture will continue to be at center stage in any foreseeable water strategy in the region.

The challenges of managing water in South Asia are largely due to increasing development pressures, resource stress, ecological insecurity (climate change), management and policy failures (IWMI 2004; 2011;David 2005). Among the problems faced are the facts that: irrigation efficiency is <40 % against an achievable potential irrigation efficiency of 60%, resulting in low cropping intensity and productivity[8]; farmers over-rely on subsidies; there is inefficient conjunctive use of surface and groundwater; trans-boundary issues exist both within and across countries; limited choices of technologies for efficient and cost-effective irrigation exist and support for R&D is inadequate; water markets are underdeveloped; water pricing is seldom observed; and even where attempts are made to charge for water, collection mechanisms are weak.

South Asian Formulation of Water policies

South Asia is host to some of the world's largest river systems, and many other, minor rivers. The types of irrigation systems and policies countries have pursued vary considerably: India and Pakistan have developed large contiguous centrally controlled surface irrigation systems; India specifically has an enormous storage capacity with approximately 4000 dams and barrages (Briscoe, 2007). Bangladeshi water policy has concentrated investments in large scale multi-purpose flood control and drainage projects (FAO 2010), while Sri Lankan policy makers have focused on alleviating seasonal water scarcity in the dry zone through large scale storage tanks and inter-basin transfers (Ariyabandu, 2008).

Trends of Investment in Irrigation

Major investments in irrigation in the region started during early 19th century and continued for over a century. More recently, government promotion of irrigation in South Asia started extensively with the Green Revolution in the mid-1960s, when it was fuelled by the introduction of high-yielding varieties of cereal grains and other agricultural productivity enhancing technologies and by soaring food grain prices which promised a high rate of return to irrigation investment. South Asian governments were further supported by Western donors who worried about food insecurity in the world's leading area for cereal production. From 1962 to 1985, irrigated area in South Asia grew at an average of 2.7% - 3% a year, which meant that it nearly doubled during that time frame. Many large dams, reservoirs, and canal distribution networks were constructed and significant investments were also made into head works, pumps, drainage roads and land leveling, all strongly supported by the World Bank and other lenders. Public spending on irrigation was at an all-time high, withmany countries in the region as well as the Word Bank allocating 50% or more of their agricultural budget to irrigation development (World Bank 1991 as cited in Barker and Molle 2004).

However, starting in the 1980s, public investment in irrigation started to decline, taking up less priority in budgets and slowly disappearing from the agenda of international development organizations. By the late 1980s, lending for irrigation by the World Bank and the Asian Development Bank had fallen to less than halfits level a decade earlier (World Bank 1995; Rosegrant and Svendsen 1993 as cited in Barker and Molle 2004) and although irrigation is now coming back into focus as part of national food security strategies as well asclimate change concerns, investment remains relatively low. (Barker and Molle 2004).

What explains these developments? First of all, grain prices declined. The expansion of irrigation combined with the spread of green revolution technology had led to a massive increase in supply, prompting developed country governments to increase subsidies to their grain producers, further boosting world supply & reducingprices. Between 1975 and 1985, cereal prices dropped more than 50% and have declined even further since then, thereby greatly reducing the rate of return to irrigation investments. At the same time, rising incomes around the globe shifted consumer preference away from cereal staples. As a result of these successes, irrigation slipped down in both domestic and donor priorities.

Second, construction costs increased, particularly because new project sites were less suitable to irrigation, further reducing the cost-benefit ratio of irrigation projects (Kikuchi et al. 2001; Svendson and Rosegrant 1994 as cited in Barker and Molle 2004). Next, operation and maintenance was gravely underfunded, shrinking the achieved returns from irrigation by as much as 44% over time (Kikuchi et al. 2003). Lastly, environmental opposition to dams grew prominent and dampened enthusiasm for irrigation. (Barker and Molle, 2004).

In general, many irrigation projects initiated during the green revolution performed poorer than expected, which played a role in discouraging further investment (Inocencio and McCornick 2008). Yet, it cannot besaid that investments into irrigation did not pay off. Many studies have documented the contribution that irrigation has made to alleviating both temporary and chronic poverty (Hussain and Hanra, 2004) and gener- ally, economic returns to irrigation projects were positive (Inocencio and McCornick 2008). With the rising importance of alternative uses of water such as for hydropower and industry, the payoff from investments in multi-purpose dams or even from adoption of high-efficiency technologies by farmers may even be larger today than in the past. The main question facing South Asian governments today is thus not whether irriga-tion pays, but how its profitability can be maximized in the long run and how both politicians & farmers may be motivated to make the necessary investments.

Surface Irrigation Policies

Different countries in the region have had varying degrees of success in pursuing larger scale surface irrigation systems, such as increasing storage capacity and transferring water from water-abundant to water-scarce areas, both of which are becoming increasingly necessary with greater climatic variability.

In 2002, the Government of India decided to launch the National River Linking Program, which seeks to transfer water from the water abundant regions to the water scarce regions of the country. The proposedbenefits include the formation of a gigantic South Asian water grid which will annually handle 178×109 m3/yr of inter-basin water transfer; construction of 12,500 km of canals; generation of 34 Gigawatts ofhydro-power; addition of 35 million ha to India's irrigated areas and generation of inland navigation benefits. (Verma, 2008) However this project has been criticized widely due to its negative environmental impacts and alternatives such as virtual water trade have been suggested (transferring virtual water in the form of food grains instead of physically transferring large quantities of water). In Pakistan, the Water and Power Development Authority and the Indus River System Authority (IRSA) are continuously working butwith little success to date to get the provinces to agree on the construction of further dams and reservoirs. Inter basin diversions are cited as future prospects to deal with the increasing costs of irrigation in Nepal.

Perhaps the biggest proof that irrigation continues to pay for the end user is the sustained expansion of the groundwater economy, which we will return to in a later section. Despite some grand plans for future development of larger-scale irrigation systems, many policies are turning to increasing the productivity of existing schemes, through a) water-saving technologies and practices, b) restoration and maintenance and c) improved management.

a) Restoration and maintenance

IFI lending of earlier decades presumed that the debtor countries would maintain facilities once built up,and so did not provide funds for maintenance. Due to poor maintenance, many of the large scale irrigation sites developed since the 1960s have deteriorated significantly, and lenders are now more amenable to financing maintenance and rehabilitation. These changes have had the effect that rehabilitation projects arenow a common sight and are often found to have higher returns than new project construction (Inocencioand McCornick, 2008).

India, in particular, has been struggling with the "build-neglect-rebuild" phenomenon, which sees the rapid decay of existing irrigation structures due to weak maintenance, and is therefore focusing new investmentson enhancing the productivity of the existing system through reforming management practices (Briscoe,2006).

One of the pioneering projects started in Pakistan in this regard was the Irrigation System Rehabilitation Project in Sindh started in the 1980s with strong donor support. The World bank, ADB and JBIC financed National Drainage Project in 1997 which completed in 2005 (Musharraf unveils plan, 2002) and large invest- ment and institutional reforms components were included. The total cost of the project was US$ 760 million. Considerable funds have also been diverted towards solving the problem of water logging & salinity through lining the canals and installation of tube wells. (FAO 2010) However, insufficient attention was paid to man- agement issues during the process. This is a topic we now turn our attention to.

b) Improved management – The role of WUAs and FOs

Till recent times, investments by governments were predicated on the fact that much of the problems with the large-scale structure were technical and hence there was a need for technical solutions. This limitedapproach did not work since much of the problems were organizational and political in nature. Failure to identify the cause of these problems has led to a major portion of the governments' investments to be directed towards the rehabilitation of the system (e.g. in the preceding case).

However, reform of irrigation management has begun to take center-stage in discussions on how to improve irrigation in South Asia. In view of the poor performance of many irrigation systems, the Bangladeshi govern- ment has recognized the importance and need for introducing appropriate on farm water management (FAO 2010) and Sri Lanka has announced that a focus of investment in the future will be on integrated water resource planning and management (Interim National Water Resources Authority of Sri Lanka 2011).

One major policy change with regards to improving irrigation management in the last two decades has beenthe devolution of power in irrigation management from the government to the water users associations (WUAs) and farmers' organizations (FOs). This transfer of power has initiated greater participation from farm- ers in irrigation management. However, the policies followed by governments differ across as well as some- times within countries (notably in India's case), and differences in state capabilities and political will to imple-ment and run WUAs and FOs also makes a general determination of their efficacy moot.

For instance, in Sri Lanka and Nepal the management of small schemes has been transferred to the WUA but the larger schemes are under joint management of the state and the WUA. In Sri Lanka, the main function of FOs is to deal with irrigation matters but they can also formulate and implement agricultural programs for their areas. The ownership of the water resources, however, remains with the government, both in Sri Lanka and Nepal. (Ariyabandu, 2008; Bhattarai, 2002) In Pakistan, the government initiated the National Drainage Program that entailed a shift in the strategic decision making policies away from the state to the FOs. This policy facilitates greater use of market mechanisms, on-farm capital investment, water allocation and O&M (Dinar et al., 1998).

In most South Asian countries irrigation is a state matter and the efforts to implement WUAs and FOs varyfrom state to state. In India, for example they range from cosmetic changes in Haryana to more comprehensive arrangements in Gujarat and Andhra Pradesh. The main responsibilities of the WUAs and FOs includeoperation and maintenance of the irrigation system, water distribution as well as the collection of water fees. Moreover, the Andhra Pradesh Farmers Management of Irrigation System Act of 1997 stipulates that officials of the irrigation department are accountable to the WUAs.

The impact of WUAs and FOs has been analyzed in various empirical studies. For instance, studies conducted in Sri Lanka (Samad, 1998) and India (Cornish, 2003) reveal that the participatory approach did not lead to a reduction in government expenditure for O&M. However, results for Nepal proved otherwise. A study conducted by Cornish and Perry (2003) found that in Nepal the rate of recovery of O&M costs was only 1.3% under schemes managed by the department of irrigation (DOI) and fee collection rate was 30%, whereas under schemes managed jointly by WUAs and the DOI the collection rate was almost 58%. In India, there is also evidence that FOs improved the quality of irrigation service provision: a study was conducted by Naik and Kulro (2000) in Maharashtra, in which farmer surveys were carried out to assess the impact of FOs under the Mula and Bhima canal schemes and results showed that 82% and 74% of the farmers, respectively, ranked WUAs as their first choice water supplier. On the other hand, in Sri Lanka, where the government formally transferred the operation and management of the irrigation system to the FOs in 1998/1999, a study in Nachchaduwa found that nearly 60% of all farmers interviewed felt that the condition of the canal system was worse after management transfer (Samad and Vermillion, 1998). Infrastructure inspections also revealed a serious under-investment in maintenance.

Most of the available evidence regarding WUAs and FOs is at the micro or scheme specific level. As statedearlier, there are too many differences in prevailing conditions, policy design, and implementation to draw firm conclusions as to the best role of WUAs and FOs in water management. However, there is broad consensus that more participative decision-making is a desirable feature of a system. Building capacities and stronger farmer groups requires a lot of time and resources, which the governments will eventually need to invest for projects to be viable and sustainable (Briscoe, 2007). In the absence of firm evidence in favor of oneset of design decisions, governments of the region would be well served by starting small, experimentingwith various different ways of designing these bodies, collaborating within and across boundaries to build collective wisdom, and scaling up successful designs gradually.

Groundwater Irrigation Policies

Whereas public investments in surface irrigation has waned, South Asian countries have experienced a groundwater boom as private investments in tube wells and other groundwater technology continue, resulting in large areas in the semi-arid regions of South Asia that depend on groundwater irrigation. Today, India is estimated to draw some 60% of its irrigation water from below the surface. In the populous Punjab region of Pakistan, it is estimated that groundwater accounts for 40% of irrigation. The total area irrigated bygroundwater in South Asia is on the rise.

Groundwater development is less restricted by topography and hydraulics than large surface irrigation systems and is better suited to private development as tube wells can be independently owned and water can be used on demand. Groundwater development has thus historically been seen as a core element of livelihood creation programs for the poor in the developing regions of South Asia, supported by subsidies for tubewell equipment and especially, pumping electricity (Shah, 1993; Kahnert and Levine, 1993).

There are few regions left in South Asia, in which there is still scope for further groundwater development. In these areas, subsidies facilitating groundwater extraction are very effective in improving agricultural production and alleviating rural poverty. Even though such regions are declining rapidly, government policies aimed at facilitating groundwater extraction persist, leading to increased stress on groundwater resources and growing concerns of the impacts of overexploitation, meaning that the rate of water abstrac- tion exceeds recharge possibilities and will ultimately lead to groundwater depletion and subsequent decline in agricultural output (Shah 2007). This policy persistence has led to socio-economic disasters such as those currently being experienced in Southern Rajasthan, coastal Saurashtra and Tamilnadu, and North- ern Gujarat, where the ecological consequences of overdraft are being fully realized, leading to abandon- ment of entire village clusters.

There is no disagreement about ideal policy interventions (our discussions of the economics of groundwateruse in Appendix A and on implementing market trade in groundwater is particularly important in this context): they include formulation and enforcement of a groundwater law, establishing unambiguous tradable property rights for water, treating groundwater as an economic good in terms of pricing, and imple- menting a licensing and permit system in order to regulate groundwater extraction. However, no Asian country has been able to adopt these measures effectively on a sufficiently large scale.

India, for example, has been working on a groundwater bill for over three decades, but has not succeeded in legislating it due to concerns about enforcement on almost 20 million water pumps spread out across the vast countryside. Instead, not only South Asian countries but also North China are still promoting more groundwater development with little or no regard to overexploitation of the aquifers. This can be partly attributed to a lack of information regarding the actual occurrence and condition of groundwater resources in the regions and partly to political and social obstacles associated with enforcement of regulatory interventions. To give an example of the latter, Sri Lanka's recent water policy has instigated a lot of controversy, which has been fueled by the sensationalist media. The locals fear that the government has sold Sri Lanka's water resources to multinational companies, and policy reforms aimed at discouraging groundwater withdrawals by increasing the initial or/and operational cost of extraction seem to confirm their suspicions (Shah,2007).

With strong community resistance to any reforms restricting water use and high direct monitoring costs, pricing of energy has become the only medium through which governments can indirectly regulate ground- water withdrawals in South

Asia and energy remains subsidized in much of the region.

Persistence of Rain fed Agriculture in Regions of High Climatic Variability

Despite the fact that most of the world's irrigated area is found in Asia, 58% of cultivated land in South Asia remains rain fed (Wani et al. 2009). In the face of increasing climatic variability, this seems worrisome; how- ever, in comparison with other regions in the world, including developed countries such as the USA, SouthAsia has a very high irrigation rate.

The reasons underlying the persistence of rain fed agriculture are manifold. Firstly, investments in irrigation are driven to an important extent by necessity such that more humid areas are less likely to develop extensive irrigation infrastructure. At the same time, public irrigation investments tend to be focused on high- potential areas, such as densely populated districts or regions located close to watercourses, major marketsand roads (Kerr 1996). Given limited state resources, this has had as a consequence that semi-arid areas which remain predominantly rainfed are some of the poorest regions in South Asia, where complimentary conditions are not always conducive to private investments in irrigation on the part of farmers (Bantilan et al. 2003). In these regions, there is a need to invest in rain water harvesting and storage technologies including natural wetlands, groundwater aquifers, ponds, small tanks, as well as micro dams to buffer against increas- ing climatic variability (McCartney and Smakhtin 2010).

From a policy perspective, this variation of investment across regions raises the question of how decisionsare made about investments by governments: if decisions are made on a measure of efficiency, it may be useful for policy-makers to consider diverting some water to "protective irrigation" of dry land crops, since this could substantially improve yields in those crops (Kerr 1996). Some studies suggest that investments into infrastructure and technology in rain fed areas have high marginal returns that out-compete additional such investments in irrigated areas as well as expansion of irrigation and call for governments to move into this direction (de Fraturier 2007, Fan et al. 2000). However, it is rare that extended areas are completely irrigatedor completely rain fed. In a study of rain fed agriculture in India, Kerr (1996) points out that irrigated and rain fed agriculture co-exist in practically every village; farmers tend to intensively irrigate crops such aspaddy, sugarcane, and horticultural crops while leaving grain crops rain-dependent. When differences in irrigation are across crops, this simply suggests that more expensive irrigation processes are reserved forhigher-value crops.

However, if access to irrigation services varies across closely-situated farms, this raises questions of equitability and efficiency, since it suggests that these differences are not driven by differences in the farms' suitability for irrigation. If investments in irrigation are a result of owners' characteristics, such as having the financial resources to invest in tube wells, this implies funds may not be being directed to the most efficient investments. There are two policy directions that emanate in such a situation: first, policy-makers must focus on removing barriers to investments at the individual's level – cheap access to credit, subsidized inputs, farmer education and extension services etc. can all be used to ensure that fewer farmers face the situation of being on a farm that would benefit from being irrigated but is not because of a removable constraint. Second, policy makers should ask the question why farmers cannot privately cooperate to irrigate in the most efficient physical way, and then find a way to share surplus. This question is discussed in detail in Appendix
B. The short answer is that there exists a problem of trust in collaborations.

Most importantly, it must be stressed that water is a finite resource and hence irrigation potential is limited. In India, for example, the optimistic estimate for total irrigation potential of 175mha represents only about 72% of existing agricultural land (World Bank 2011b, ADB 2009). Given this, it is vital to identify how existing water resources can be used both efficiently and equitably as well as how productivity in rainfed areas can beincreased to meet future food demands.

Water-Use Efficiency Policies

There has been a recent move in South Asia towards subsidizing new water efficient irrigation technologies. Much of the focus of these subsidies has been on micro irrigation technologies such as the drip and sprinkler systems. Subsidies and options for financing from organizations and government schemes increase the profitability of investing in micro-irrigation, which makes a crucial difference in adoption by poorer farmers. (IWMI, 2006) Thanks to government subsidies, drip irrigation is expanding rapidly in India. In Pakistan pilot projects have demonstrated the technical benefits of these techniques and the government is now directing subsidies in the same direction. These subsidies have concentrated at the farm rather than the surface water conveyance system level. (Faurès, 2007) Nepal has also indicated that most of the future investments asenvisaged in the five year development plans are directed towards the widespread use of appropriate technologies to increase agricultural production. (FAO, 2010) .

The other aspect of improving on-farm utilization of water is proper irrigation timing and planning, which is often undermined by a lack of knowledge on crop water requirements and the like on the part of the farmer. For example, in Pakistan, due to a lack of research and extension services, Pakistani farmers have little under- standing of the most productive applications of water during crop-growing cycles which has led to lot of water wastage in the system. All these problems should be the focus of any future investments undertaken by the governments in this sector (Briscoe, 2007).

Economists widely advocate the use of water trade in markets as a means of efficient allocation. Prices in a well-functioning market reflect both the marginal cost of supply and the marginal value of use, a feat that is not possible for even the best central planner to match. When water can be traded, it carries an inherentopportunity cost that creates incentives to conserve water, use it efficiently, or trade it away to higher-valueusers.

Tradable water rights encourage investment in water saving technologies because investors benefit monetarily from the savings (Rosegrant and Gazmuri, 1995). They also allow users to engage in activities requiringlarge quantities of water provided the surplus created is large enough to buy the additional water needed. In the same way, changes in crop prices, demand patterns, and relative efficiency of water use can all be responded to with maximum flexibility, resulting in efficiency-enhancing water reallocations. This flexibility that the ability to trade water provides is expected to become increasingly important for cross-industry allocations too, as non-agricultural demand for water grows, e.g. through continued urbanization.

Moreover, since voluntary exchange in a well-functioning market requires that any reallocation of water only occur with the original water user’s consent, the user is empowered and cannot theoretically be worse off than without trade (since any exchange that makes him worse off will be overruled). However, as the discus- sions below demonstrate, problems can arise in at least two ways: due to market failures, or because the prerequisites needed to support functioning markets do not exist.

Markets in surface water systems

One of the countries with the most advanced water markets globally is

Australia, which offers transferable water entitlements within the Murray Darling Basin (Qureshi, Shi and Qirbi 2009). The Australian water industry was divided and decentralized in the early 1990s; Murray Irrigation, for example, is a separate licensed entity with its own board. The system also allows water trade between states. Subsequent tariff reforms have ensured that consumers now pay according to the quantity and the efficacy of their water use. The National Water Commission (NWC), as both regulator and investor, has been the system's backbone. This model pre-supposes that agriculture is a commercial sector, which is far from the reality in most of South Asia'sdeveloping countries.

Another example cited in favor of water markets, and one that is argued to be more relevant to South Asian countries, is Chile's experience with transferable water use rights, which it introduced in 1981. A study conducted on these reforms by Heame and Easter (1995) found that the transfer of water-use rights produced substantial gains from trade, and that WUAs played an important role in facilitating the marketreallocation of water, especially in the Limari valley. On the other hand, Romano and Leporati (2002) argue that the reforms led to higher inequality in access to water as farmers with lower social and human capital and little access to information were in a weak bargaining position. In 2005, the Chilean government passeda reform to the Water Code, amending it to give the government more control over the social & environment effects of water trade. In the end, although Chilean water trade has been successful in allocating water fromlower to higher-value uses in some instances, water markets by no means function all over the country;many farmers are not aware of the Water Code (1997), do not have the resources needed to function in aformal market or have not even registered their water rights (Bauer 1997). It is also important to note that water rights existed in Chile before the introduction of the Water Code, which makes pre-existing conditions for functioning water markets much more favorable than in South Asia, where water rights are not as established a concept. For a water market to function certain preconditions have to be in place; well-defined water rights, measurement devices and routines, enforcement and sanctioning mechanisms as well as specifications concerningreturn flows. (The World Bank 1999) We will discuss these in detail in a later subsection.

Markets in Groundwater

Groundwater markets are fundamentally different from their surface water counterparts. In sophisticated surface water trade, buyer and seller may be very distant from each other and interact only through thewater market, while trade of groundwater is typically localized and personal, and infrastructure require-ments are correspondingly far smaller. Therefore, although South Asia may still face significant hurdles to surface water markets, groundwater trade is already commonplace in the region.

Tubewell water sales have, for example, become a profitable enterprise for small farmers in Uttar Pradesh in India (Shankar, 1992) and even for the landless under programs sponsored by several NGOs in Bangladesh. In Gujarat, India, water markets are highly advanced and farmers make substantial private investment in water pumps and underground pipeline networks. This generates a high degree of competition amongst sellers of water. (Dinar, Rosegrant and Meinzen Dick 1997).

For suppliers, the major benefit cited of groundwater trade is that by selling water to other farmers, tubewellowners can use a higher proportion of their well capacity than they would on their holdings alone and realize a higher return on their initial investment. On the demand side, groundwater markets increase access to irrigation water, especially among farmers with small or fragmented holdings and those without their own wells, thus increasing equitable access to water as an input. Shah (1991) argues that the resultant expansion of irrigation has led to increased cropping intensity and agricultural labor demand, ultimately benefitting the landless and those who rely on wage labor for household income, and lowered water tables in water-logged areas.

As effective as their positive effects may be, unregulated markets in groundwater are subject to a fundamental flaw: trade of surface water is typically separated from production decisions and the decision is essentiallyof how to allocate a fixed amount of water to rival uses. On the other hand, in groundwater markets the selleris typically the owner of a pump who can sell water that he would otherwise not have pumped[19]. This adds a significant possibility of overexploitation absent in surface water markets.

The ability to trade groundwater exacerbates the Commons problem (studied in detail in Appendix B) thatthe non-excludable nature of groundwater exposes it to, especially with the prevalent ownership rule of First Possession. This has, predictably, already led to overexploitation of groundwater resources in some areas where groundwater markets exist, beyond the possibility of recharge. In short, groundwater markets, by increasing access to water pumping, magnify both the beneficial and harmful dynamics that pre-exist in groundwater pumping.

Preconditions and limitations of markets

As discussed earlier, for all but the most rudimentary and inefficient trade, transactions must be under- pinned by clearly defined water rights, verifiable measurement and credible enforcement. This presentsmany difficulties.

First, since any formal market in either surface or groundwater requires clarifying initial rights, this entails apportioning rights according to a principle or rule. Since rights are rival, this implies that there will be winners and losers. In groundwater, for example (and as discussed in the appendix), a rule of tied ownership (rights to water linked to property rights over land) favors those who have not yet developed their water pumping capacity over those who have, when compared to a rule of first possession (whoever pumps ownsthe water). In surface water, allocating rights according to current or historical usage, linking rights to amount of land, or assigning flat rights to each person all impose different costs and benefits on different people. Support for a particular rights apportionment principle will depend on these expected costs andbenefits that people face.

Moreover, the existing valuations of land already incorporate formal or informal expectations of water apportionment (or prevailing sense of entitlement to water rights). New laws will correctly be perceived by some existing land owners as expropriating value. An added complication arises when flows are variable: if rights must be conceptualized not as claims to an absolute amount of water, but a share of whatever water exists, this clouds expectations. Finally, the new rules and institutions may clash with preexisting informal rules of resource allocation that grew out of local traditions and culture (Griffin, 2005). Resistance is almost certain because such an allocation of rules is disruptive both economically and culturally.

By reducing wastage, incentivizing water saving, discouraging overexploitation and encouraging reallocation from low-value to high-value users through trade, water markets that meet the necessary preconditions improve economic efficiency. However, many of the benefits[21] are dispersed across a large group, and accrueto individuals with little say in the political process ex ante. These people may also be rationally ignorant of these benefits. It is therefore conceivable that those enjoying cheap access to water, effectively ridingroughshod over rival (likely lower riparian) claims, will lobby to preserve the status quo. The precondition that initial assignment of property rights is completed is therefore one of the biggest hurdles to the functioning of water markets. As noted previously, Chile had existing water rights at the time water trade was introduced.

Similarly, Australia not only had water rights, but the idea of trading water like other inputs was well- established because agriculture was already viewed culturally as a commercial, not a social activity. South Asian countries would do well to set their sights on the lower but more difficult goal of establishing unam- biguous property rights before water trade is promoted.

Since apportioning water rights places burdens on some users, it should be done as part of a broader pack- age of reforms that carries other benefits that may be compensatory for those whose water rights may have been affected. The western experience with water markets was successful because it was coupled with agricultural policies that promoted less water consuming crops, and carried out subsidization of irrigation equipment to reduce the burden on farmers. (Berbel, Calatrava and Garrido, 2005) Employing such policies in developing countries of South Asia in conjunction with the market may be ambitious.

Another separate problem that needs addressing before water markets can be established in South Asia isthe improvement of the infrastructure and administration of water measurement. Currently, water measurements are mostly done at the canal or distributaries, not the farm level, and this complicates efforts to trade because claims of withdrawal levels are not verifiable, and the enforcement of withdrawal rules thus under- mined. Improving measurement is not merely a technical problem, but is also made difficult by resistance from powerful local elements, corruption and formal political resistance, since overdrawing is prevalent.

Moreover, even if these requisite preconditions are met, while there are clear benefits from using market mechanisms, there are also important limitations that need to be addressed in practice. First, bilateral volun tarry trade is efficiency-enhancing if there are no externalities, but not otherwise. If a farmer's decision to buy water implies congestion of water channels, environmental degradation or unsustainable abstraction from a common aquifer, the rules of trade must regulate and account for these effects. Unregulated, a system of tradable property rights impedes the development of effective river basin planning and environmental and ecological protection. (Dellapena 2008)

Second, while voluntary trade is considered welfare-enhancing in most environments, inequalities of social or human capital, and little access to information can weaken an agent's bargaining position. Ostensibly voluntary trade can be exploitative or even fraudulent if the market participant cannot read or interpret regulations properly. A complementary condition for market development therefore is the development of com- munity organizations to advise, educate, and manage water allocations. (Rosegrant, 1994)

To sum, while market allocation can be clearly efficiency enhancing in some cases, the development of markets cannot proceed in an isolated fashion from the real-world institutional and technological con- text of developing country irrigation. We discuss this issue more broadly in the next section.

Problems in South Asian Water Policies

Over the course of this document, we have described South Asia's experience with past and current waterpolicies. This section summarizes the preceding discussion and documents problems. As a water-stressed region, South Asia clearly needs to harvest more water, and use existing water resources more efficiently.

Problems of water use

Surface water is, for reasons discussed in Appendix B, primarily managed by the state and involves large investments because governments need to construct dams of both micro and large scale. However, both foreign anddomestic investments in irrigation have declined. As already documented, donor support for irrigation projects has greatly reduced over time. Furthermore, governments are faced with ever increasing demands on their budgets, further squeezing money allocated to agriculture and specifically irrigation. For a sector asimportant as irrigation, South Asian countries need to look inwards for investments, and not be dependent on aid.

If the irrigation investments are truly beneficial, i.e. social returns net of costs are positive, it is not action but inaction that states can ill afford. Developing countries in general, and South Asia in particular suffers from the problem of not ranking potential investments across different sectors by favorability in a cost-benefit analysis. In making investment decisions, governments can seldom refer to evidence that money spent on the proposed project is likely to provide greater socio-economic returns in this use than in any other. As a theoretical example, it may be beneficial to reduce de-silting of canals in order to invest resources in providing protective irrigation in remote dry lands, but the decision-making process for such investments will often be independent.

Another problem is the lack of farmers' involvement, both during initial planning and implementation of projects and in operation and maintenance. Irrigation projects that have some farmer contribution tend toperform better than solely government managed systems, yet most water development for irrigation in South Asia has been implemented using a top-down approach with little involvement of farmers. As a result, farmers often do not feel like they have a responsibility in maintaining the projects.

While it is recognized that farmers must more actively contribute to the operation and maintenance of irriga-tion infrastructure to increase its longevity and efficiency, further work is needed to understand how the state can incentive such contribution: the evidence on water user association (WUA) management varies case-to- case, and more investigation is needed to understand what does and doesn't work. While South Asian govern- ments have begun encouraging WUA, progress is slow and effectiveness far from guaranteed. (Inocencio andMcCornick, 2008).

On the other hand, there is broad agreement that water charges in a centralized system should be volumetric if possible, and water fee collection needs improvement. Surface water is currently nearly free, and groundwa- ter volumes abstracted by farmers virtually uncontrolled. This provides farmers few incentives to switch to high-efficiency technologies and results in water shortages at the tail end of watercourses. Meanwhile, farmers in other areas are paying as much as one-third of their produce for pump irrigation services, suggesting that the value of on-demand irrigation far exceeds what is currently contributed by canal water beneficiaries (Shah 2007). It is also known that water rights trade can improve efficiency by placing an opportunity cost on more items, and incentivizing water savings. Relatively little is known, however, about how the market will function when exchange is voluntary but when market participants have unequal human capital, or when information is weak.

Finally, since surface water and groundwater are both affected by the resistance to technical, institutional and political reform, we will defer the discussion of this to the end of this section. Having discussed the issues ofirrigation at the system level, we now turn to issues at the level of the farm.

We have already discussed how surface water prices are too low. Meanwhile, groundwater pumping is largelyunmonitored in South Asia, and hence goes untaxed. Energy for pumping is even subsidized in some signifi-cant cases, the most important of which is India, where groundwater levels are fast shrinking. (Rodell,Velicogna and Famiglietti, 2009) The biggest hurdle to efficient and careful use of water is therefore that farm-ers have no direct economic incentive to be efficient or careful in how they use water. Ironically, a water- stressed region, which as a collective would happily conserve water as a resource, is unable to incentivize the individual farmer to turn the tap off.

The weak incentives to conserve water that do exist are largely indirect and insufficient – over-application of water at the farm can create water drainage issues, or reduce the effectiveness of fertilizers and pesticides. Similarly, frequent power outages can increase water application times and unlined water channels absorb too much water as it lays dormant when a brownout occurs – farmers respond by lining water channels to deal with these issues, and water application efficiency increases as a side-effect.

Instead of bemoaning the tragedy of the commons, however, pragmatic policy-makers would do well to embrace it and use it to organize their thinking for at least the short to medium terms – the pitch to a farmer for saving water cannot be a vague appeal to the common good, but a demonstration that doing so is likely tocarry real economic benefits, in the shape of better produce, fewer pests or savings in other inputs. Every new technology that is a candidate for a widespread adoption drive should be evaluated in terms of whether itcarries these benefits along with the water-saving. No matter how attractive in theory, a technology that saves a lot of water but carries none of these ancillary benefits is unlikely to be widely adopted.

Obstacles to private take-up/PPP

The question that then follows is: if a technology carries economic benefits for farmers, why don't they adoptit without government support? First, it is argued that smaller and medium-scale farmers are often cash- strapped and unable to make large initial investments into irrigation infrastructure on their own. As the initialcost of much irrigation infrastructure, from water channels to drip irrigation systems, is high, this creates afinancing gap for farmers.

Second, and relatedly, adopting new technologies can be very high risk - farmers are unlikely to invest in new technologies without having witnessed their productivity and reliability first hand.

Third, there is a lack of technical know-how in the private sector – even if a farmer has the money and therisk-taking appetite to want to adopt new technology, there are often few private firms accessible that have the ability to install, calibrate and maintain some of the newer technologies.

Finally, since the adoption of many newer technologies also requires a move to different, unfamiliar types of produce, the farmer needs access to all the associated resources involved, such as different inputs, transport,and access to markets. Any link missing in this chain can raise the costs of switching to the point of rendering the move infeasible (Beggs, 1989).

Policy Options

The government can react to reduce each of these obstacles but must do so carefully. First, it must decide what not to do. It should not, for example, react to the financing gap of the private sector by providing subsi-dies for this reason, because it faces a financing gap of its own and needs to apportion its resources to theprovision of public goods. Moreover, subsidies tend to be politically hard to remove once they are in place, and rent-seeking behavior can render their long-term effectiveness dubious at best.

Instead, the causes of the credit constraints have to be recognized and efforts made to remove them. A majorreason smaller farmers in particular do not have access to credit is the absence of sufficient collateral because property rights are either ambiguous or because rival claims to property cannot be properly resolved in courts. The solutions to this problem lie in long-term improvements in the enforcement of contracts, in reso- lution of court cases, and their effective enforcement, which are all beyond the scope of any irrigation policy.

Second, in the short term, both the fact that a new technology is high-risk for an adopter and that there is a lack of private-sector providers of support can be tackled by attempting to increase relevant knowledge in the economy. It is fairly uncontroversial that education has positive externalities, and the first policy that can be adopted is to set up or expand farmers' teaching facilities. Besides general agricultural education, farmers can learn about more efficient irrigation technologies as well as improving farming techniques. A similarcurriculum, but at a more basic level can be used in awareness campaigns and extension services.

Possibly even more important than these, however, is the demonstration of the potential success and efficacy of new technologies. As noted earlier, farmers tend to be risk-averse when considering the take-upof innovative tools and processes, since these require a sure cost up front, but have the very real possibility of failing. The government can build a record of success stories and develop material that explains to farmers how others with similar backgrounds, education and resources have adopted the technology. Moreover, the government can take the lead in adapting the technology to the region if necessary. There are multiple ways to increase water use efficiency, including reliance on irrigation scheduling, various new mechanisms for irrigation management to increase precision, etc. Some of the challenges for research in extension is to take advantage of new knowledge in technologies and develop new ways to improve water use efficiency. Furthermore, changes in prices of energy and commodities should also trigger effort to improve water useefficiency.

Besides developing and transmitting a technical literature, the government can try to seed awareness bytargeting subsidies and supporting technical help for a small number of farmers specifically in areas where success stories do not exist and lower subsidies as technology take up increases in the region. As mentioned in Appendix A Punjab province in Pakistan followed this model very successfully with laser land leveling in the recent past. The literature suggests that subsidization and adoption of modern irrigation technologies should be done selectively when they increase water holding capacity or the precision of irrigation signifi- cantly, which may occur in locations with sandy soil or uneven land. (Schoengold and Zilberman, 2007) Adoption should also occur with higher value crops so the gain in net value can pay for the investment.

In the medium-term, the government can also help address the problem of high switching costs. The government can help develop the value-chain for the innovative product. There may be a need to develop public goods, such as infrastructure. In some cases simply subsidizing the move would be sufficient. It may even be that simply signaling the presence of a subsidy serves to reassure dispersed early adopters that their private adoption of the technology will not be unilateral, but part of a wider adoption that will mitigate the risk of a failure to build momentum.

In the long-term however, prices must increase to encompass all costs of production. If handled naively, such a policy will encounter severe political backlash, since it creates losses for some (those enjoying inefficiently low prices now). Close attention must be paid to the sequencing and implementation of any price increase. Awareness of the reason for price increases must be created: the public must understand both the extent of the water scarcity arising in the region, and the fact that the point of the increase is not to extract a region or subgroup’s surplus, but to engender good decisions on the margin. A credible public commitment should be made to invest the money collected back into the local area, if political resistance is to be averted or tackled successfully[24]. Before prices are increased, credible alternatives should be provided, in the form of subsidized access to water saving technologies, and it should be announced that the subsidies are explicitly with the intent of ensuring that those bearing the costs of higher water prices will not suffer.

For surface water, it is then a simple matter of increasing prices gradually. For groundwater, the complication is that there is no feasible infrastructure to directly monitor usage and price water extraction. In the short term, these are largely intractable, and the workaround of possibly taxing the energy source (electricity or diesel) needs to be studied. Careful study of existing electricity usage, the possibility of arbitrage, and the calculation of an appropriate price is required, but it is likely that the problem of unintended consequences will remain, and there is a real danger that in trying to force water use efficiency through such a blunt policy instrument, energy use efficiency will be harmed. In the longer term, as discussed earlier, any move to directly regulate extraction requires monitoring, which requires both technological and institutional advances.

Obstacles to policies

Echoing our earlier question about farmers' lack of technology take-up, why doesn't the state pursue thepolicies outlined above? South Asian states are financially weak: they often do not have the resources topursue infrastructure or technological improvements or provide wide and sustained subsidies. Technical capacity is low: governments lack dedicated and capable personnel, are beset by bureaucratic inefficiency and corruption, and further lack reliable data and physical mechanisms for controlling water use.

The institutional frameworks governing irrigation in South Asia are weak: over the past half century, irrigated area has grown faster than the institutions needed to regulate this growth (Barker and Molle 2004). There isa confusion about areas of responsibility and policy options and failure to predict consequences. The prime example of this is the explosive growth in groundwater exploitation in South Asia over the past 50 years and the entailing resource degradation. Groundwater rights remain undefined. In many cases governments have subsidized electricity used to power water pumps. Groundwater depletion is now a serious threat for many regions in South Asia and can only be addressed through improved resource management on the parts of government as well as by governments supporting the conjunctive use of surface and groundwater to facili-tate optimal flexibility in delivery combined with sustainable recharge of underground aquifers (Shah 2007).

Lastly, it is important to also consider the incentives that government officials face in implementing irrigationpolicy, and the role of politicization of decisions. For example, Pakistan's four provinces are in deadlock over approving the construction of new dams because downstream provinces do not trust that they will receivetheir fair share of water and because strong interest groups in the downstream provinces benefit from reces- sion agriculture as a result of floods, which will be reduced if new storage is constructed. This demonstrates that provincial government interests do not always align with national interests in water resource develop- ment. Similarly, on the lower end of the bureaucracy, publicly commissioned irrigation managers may notfind their incentives aligned with those of their clients, the farmers (Mukherji et al. 2010). Furthermore, governments may seem myopic and neglect the upkeep of existing irrigation schemes because they knowthat donors are more likely to fund rehabilitation projects than simple maintenance programs (Kikuchi et al. 2003). A more detailed discussion of the political economy underlying irrigation in South Asia is provided in Appendix C.

Some of the problems discussed here can clearly be worked around – substantial existing subsidies suggest that financing is likely not a concern for any but the largest infrastructure projects; technical capacity is low but can be slowly increased by providing enough rewards to attract foreign trainers; government officials' incentives can be weak, but this is true all across the world – competent oversight, monitoring, and a system of rewards and punishments can make dramatic improvements.

The harsh reality, however, is that South Asian states have an entrenched track record for fundamental and consistent failure to provide service delivery to their citizens. The most fundamental consequence of this is that security of life, property and contract is weak. Political upheaval, such as experienced most notably by SriLanka in the past and Pakistan in the present immeasurably harms economic activity. The danger of theft or destruction disincentivizes investment in tangibles. The inability to write and contest contracts means thatpartnerships are mostly informal and are limited and bear greater costs as a result. These problems have nothing to do with irrigation policy – they are law enforcement subjects – but they have everything to do with whether private actors engage in the sort of dynamic work needed to improve irrigation outcomes, as well as outcomes in every other sphere of economic activity. They also mean that inputs of the highest importance, such as an educated workforce, are largely absent. This means both a less dynamic private sector and a less sophisticated and internally competitive civil service. The bottom-line then is that we must focus our attention very carefully on solving the issue of security and service delivery. To focus on advanced irriga- tion policies at the expense of these will just be rearranging the deck chairs of a sinking ship.

Conclusions

At its core, the commercial allocation and use of water can be understood in terms of its similarities with the allocation of other economic goods with dispersed benefits and the requirement of relatively high initial investment. The realities of operating in a third world context add other relevant considerations.

Water is a natural resource, but one that often requires the building and maintenance of complex and large- scale transport, storage and distribution systems. The provider must therefore often be a large enterprise,and often this is government. As a natural resource, states must come to decisions about the initial assign- ment of property rights, and then be ready to back these decisions with fair enforcement. These rights mustinclude provisions for the Human Rights to water, but water beyond that required for a person's healthyfunctioning can be treated as commercial.

In South Asia, there are two-pronged pressures on the efficient provision of water. On the one hand, there is the strong need for good provision because of large, poor and untrained populations. Governments need totake the lead in research, education, extension and training.

On the other hand, these countries carry theadditional burden of, generally, old and inefficient infrastructure. Much of South Asia currently evidences the gradual decline of institutions inherited from a colonial past or transplanted in through foreign aid.

Much can be done in the short term and at the level of the farmer to alleviate the water stress developing across the region. In the long term however, irrigation and water use efficiency will evolve in each country asthe direct result of its progress towards political maturity and good governance.

2. Concept and Principle

2.1 Issues and Problems of irrigation development in Terai and hills

A. Natural

- Less water during winter and spring season.
- Fragile Geology–Landslide in hill irrigation system.
- Excessive Siltation–Mainly in Terai irrigation system.
- Climate change- Design life सम्म नचल्ने, चाडै Rehabilitation गर्नुपर्ने ।
- O&M: मा Sustainable भएनन् । सरकारी बजेटमै भर परे ।
- Return राम्रो भएपनि कृषि व्यवसायीकरण र विविधिकरण हन सकेन ।

B. Technical

- Lack of storage Projects.
- Inefficient use of irrigation water.
- Lack of repair and maintenance.
- Lack of technological innovations.

C. Management

- –Lack of proper operation and management.
- Excessive mining of river bed because of over extraction.
- –Water pollution
- Lack of operational staff management incentives.
- Frequent transfer of personnel.

D. Design and Construction Short Comings

- Lack of sufficient hydro–metrological data.
- Design deficiency
- In 2050 B.S.there were 16000 m3/second discharge at Bagmati headworks site but the design discharge was taken only 11000m3/second.
- Tinau Irrigation project was washed out at just one year after the operation.
- Site problems occurred in most of the Terai Irrigation system.Headworks of Sunsari Morang Irrigation Project -SMIP)_was shifted towards upstream side because of the silt problem
- Canal Syphonat Budhi Bridge of SMIP was collapsed during the monsoon flood of fiscal year 2074/75.

- Headwork of Chanda Mohana Irrigation System has seepage/artesian problem due to which it is difficult to maintain pond level.
- Battar and Chitwan lift Irrigation Project are not functioning at their design capacity.
- Many Hill Irrigation Systems are in defunct situation because of Landslides.

E. Financing

- Lack of budget allocation
- Long gestation period
- Donor commitment is not continued till the completion of the Project.
- Land Acquisition problem for new irrigation project.

F. Social

- Land fragmentation, Absentee landholdings
- Aversion of youth from agriculture.
- Irrigated land used for non–agriculture purpose.

G. Agri Services

- Lack of extension services
- Lack of development of market mechanism.
- Credit, seed, and fertilizer availability.
- **H. Policy and Legal Issues**
- Implementation of land use policy.
- Implementation of irrigation policy.

I. Institutional Issues

- Ineffective irrigation institution.
- Institutional Capacity of WUA has not been strengthened.
  - Political polarization.
  - Lack of transparency.
  - Lack of good governance– accountability, consensus.
  - lack of institutional development activities
- Lack of coordination between agencies at different levels.
- Absence of central irrigation development offices in each province.
- Inadequate human resources.

J. Political Issues

- Political interferences in project selection.
- Compliances of Koshi, Gandaki, Mahankali Treaty.
  - Narayani Irrigation project is not getting agreed amount of water.
  - Delay in implementation of Mahakali Irrigation Project
- Lack of Political stability and consensus.
- Political polarization among beneficiaries.

2.2 Design, philosophy and approaches to different types of irrigation system development

PROCEDURE FOR DESIGN OF IRRIGATION CHANNELS

A canal system includes the main canal, branches, distributaries, and watercourses. Main canals, branches and distributaries are owned, constructed, controlled, and maintained by the government. Watercourses are owned, constructed, controlled, and maintained by the irrigators. Sometimes, the watercourses are constructed by the government agencies also. Their maintenance is, however, the responsibility of irrigators.

The main canal takes its supply from the source directly. The source may be either a river or a reservoir. There may be several branch canals and/or distributaries taking supplies from the main canal at various points along the main canal. A branch channel, similarly, feeds distributaries which take off at suitable locations along the branch channel. A distributary feeds watercourses which take water directly to the field. The discharge and size of irrigation channels progressively decrease from the main canal to the watercourses.

Layout and Discharge Requirements of Irrigation Channels

The layout of irrigation channels should ensure equitable distribution of water with minimum expenditure. It is, obviously, advantageous to align all irrigation channels on the watershed. This will ensure least expense on cross-drainage structures and cause gravity flow on either side of the channel. In hilly regions, the channels are aligned along the contours. Alignment of irrigation channels should be such that there are very few curves. However, if the channel curves are unavoidable, the radius of curvature of these curves should be as large as possible. For large channels, the minimum radius of curvature should be about 60 times the bed width of the channel. In case of smaller channels, the minimum radius of curvature of the curves should be about 45 times the bed width of the channel.

The maximum discharge to be carried by an irrigation channel at its head to satisfy the irrigation requirements for its command area under the worst conditions during any part of a year is said to be the designed full supply discharge (or capacity) of the channel. The water level in the channel when it is carrying its full supply discharge is termed full supply level (FSL) and the corresponding depth of flow is called full supply depth (FSD).

After fixing the layout of all irrigation channels (*i.e.*, canal system) on a map, the total command area proposed to be irrigated by each of the channels of the canal system is estimated. Knowing the crop requirements and intensity of irrigation, and allowing for seepage and evaporation loses, the amount of the discharge requirement at various points of the canal system are determined. Obviously, this requirement will progressively increase from the tail end of a channel to its head reach. Accordingly, the size (*i.e.*, width and depth) of a channel will progressively decrease form its head reach to its tail end. The size of a watercourse is, however, kept uniform.

Longitudinal Section of Irrigation Channels

Longitudinal Slope

An irrigation channel flowing in an alluvium can be designed by any of the methods discussed in Secs. 8.4 and 8.5. The slope of the channel would thus be known. However, it is convenient to choose a slope close to Lacey's regime slope [Eq. (8.35)] for the first trial design. This may need modification subsequently. The slope of an irrigation channel is governed by the country slope too. If the ground slope is more than Lacey's regime slope (or the slope satisfying the critical velocity ratio and bed width-depth ratio, or that calculated on the basis of other methods), then the slope of the channel bed is kept as the required slope (*i.e.*, the calculated one) and extra fall in the ground level is compensated for by providing vertical falls in the channel. On the other hand, if the required bed slope is more than the available ground slope, the maximum allowed by the ground slope is provided. In such cases, it is advisable to prevent the entry ofcoarse sediment at the head of the channel (see Example 8.7). Usually, ground slope is steeper than the desired bed slope and falls are invariably provided along the channels. The slope of reservoir-fed channels, carrying water with practically no sediment, may be determined by the consideration of the maximum permissible velocity.

For convenience, the slope is kept in multiples of 2.5 cm/km. The advantage of such values is that the drop in the bed level for every 200 m would be 0.5 cm which can be easily read on a levelling staff.

Full Supply Level (or Full Supply Line)

Since the channel has been aligned on the watershed, the available transverse slopes would be sufficient for flow irrigation provided that the full supply level (FSL) is slightly above the ground level. The full supply level of a channel should, therefore, be about 10-30 cm higher than the ground level for most of its length so that most of the command area of the channel is irrigated by gravity flow. It is not advisable to keep the FSL higher than the isolated high patches of land existing in the command area. These may be irrigated either by lifting water from the watercourses or from the upstream outlets, if feasible. Too high an FSL would result in uneconomical channel cross-section of higher banks. Besides, the available higher head would increase the seepage loss, wasteful use of water by the farmers, and the chances of waterlogging in the area.

The FSL of the offtaking channel at its head should always be kept at least 15 cm lower than the water level of the parent (or supply) channel. This provision is made to account for the head loss in the regulator and for the possibility of the offtaking channel bed getting silted up in its head reaches.

Canal Falls

When the desired bed slope is flatter than the ground slope, as is usually the case, a canal fall has to be provided well before the channel bed comes into filling and the FSL becomes too much above the ground level. The location of a fall depends on a number of factors. It may be combined with a regulator or a bridge for economic benefits.

The alignment of another smaller offtaking channel may require that the fall be provided downstream of the head of the offtaking canal. This will ensure that the FSL of the offtaking channel is sufficiently high to irrigate its command area by flow irrigation. One may provide either a larger number of smaller falls or a smaller number of larger falls. Relative economy of these two possible alternatives should be worked out and the location of falls will be, accordingly, decided. The distributaries and watercourses offtaking from the upstream of a fall may easily irrigate some areas downstream of the fall. As such the FSL in the channel may be allowed to remain below ground level for about 200-400 m downstream of a fall.

Balanced Earthwork

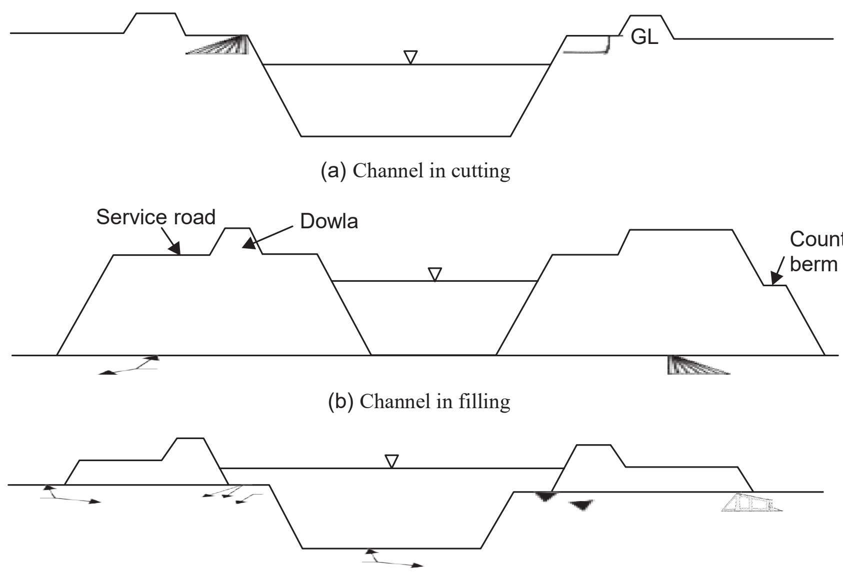
The longitudinal section of a channel should be such that it results in balanced earthwork which means that the amount of excavated soil is fully utilised in fillings. This will minimise the necessity of borrowpits (from where the extra earth needed for filling will be taken) and spoil banks (where the extra earth from excavation will be deposited). The earthwork is paid for on the basis of either the excavation or the filling in embankments, whichever is more. A channel in balanced earthwork will, therefore, reduce the cost of the project. For the small length of a channel in the vicinity of a fall there is always an unbalanced earthwork which should be kept minimum for obvious economic reasons.

Evaporation and Seepage Losses in Canals

When water flows in an alluvial channel, some water is lost due to evaporation from the water surface and seepage through the bed and banks of the channel. These losses are termed transit losses or canal losses. The transit losses are calculated at the rate of about 3 m3/s per million square metre of the exposed water surface area. These losses may be of the order 10 to 40 per cent of the discharge at the head of a channel in case of large channels.

Cross-Section of Irrigation Channels

An irrigation channel may be in cutting or in filling or in partly cutting and partly filling. When the ground level is higher than the full supply level of the channel, the channel is said to be in cutting [Fig. 8.8 (a)]. If the channel bed is at or higher than the surrounding ground level, the channel is in filling, [Fig. 8.8(b)]. When the ground level is in between the supply level and the bed level of the channel, the channel is partly in cutting and partly in filling [Fig. 8.8 (c)]. A channel section partly in cutting and partly in filling results in balanced earthwork and is preferred. Various components of cross-section of an irrigation channel have been discussed in the following paragraphs.



(a) Channel in cutting

(b) Channel in filling

(c) Channel partly in cutting and partly in filling

Fig. 8.8 Typical cross-sections of irrigation channel

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 earthfill should not behave differently from an earthfill compacted sufficiently and having the same characteristics as those of the natural earthfill. 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, as recommended by the Central Water Commission of India, are given in Table 8.6.**Table 8.6 Side slopes for unlined channel**

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

Berms

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 to facilitate drainage.

Because of irregular changes in the ground level and regular changes in the channebed level, the depth of cutting d_1 and the depth of filling, $(d_2 - d_1)$ (Fig. 8.9) 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 $(H) : 1(V)$ is the side slope in cutting and $r_2(H) : 1(V)$ 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 Fig. 8.9, 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 bermwidth is equal to $(r_2 - r_1)d_1$.

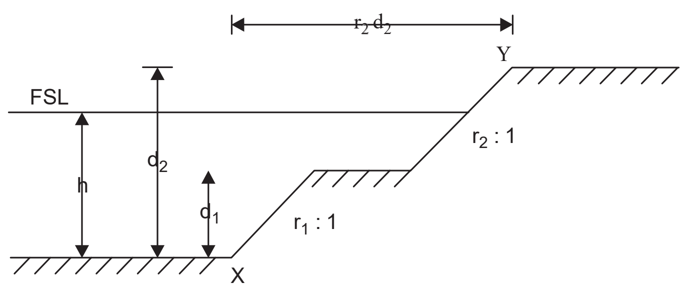


Fig. 8.9 Berm

For usual side slopes of 1.5 $(H) : 1\ (V)$ (in filling) and $1(H) : 1(V)$ (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 berms, an impervious lining is formed on the banks. This helps in reduction of seepage losses. In addition, the berms protect 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.

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, land slides, 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 varies 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.

The Central Water Commission of India (CWC) has recommended the value of freeboard as given in Table 8.7.

Table 8.7 Freeboard of irrigation channels as suggested by CWC

Discharge (m^3/s)	up to 0.7	0.7 to 1.4	1.4 to 8.5	over 8.5
Freeboard (m)	0.46	0.61	0.76	0.92

Alternatively, Table 8.8 may be used for the estimation of freeboard.

Table 8.8 Freeboard in irrigation channels (15)

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

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

$$F = \sqrt{CD}$$

Where, C is a constant varying form 0.45 (for discharges up to 0.07 m^3/s) to 0.76 (for discharges greater than 85 m^3/s) and D is the water depth in metre.

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. This has been discussed in Chapter 15. 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 very 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 Fig. 8.8(*b*), is provided. Alternatively, the outer slope of the bank is flattened.

The Central Water Commission has recommended bank widths as given in Table 8.9.

Table 8.9 Bank widths in irrigation channels

Discharge (m^3/s)	*Top width of bank (m)*
less than 0.28	0.92
0.28 to 1.4	1.22
1.4 to 4.2	1.50
4.2 to 10.0	1.83
10.0 to 14.0	2.44
14.0 to 28.0	3.66
28.0 to 140.0	4.58

Service Road and 'Dowla'

For proper maintenance and inspection of irrigation channels, service roads are provided on both sides of main canals and major branches. In the case of smaller branches and distributaries, a service road on only one side (usually, the left bank) is provided. The inspection road on main canals is usually about 6.0 m wide and should not be smaller than 5.0 m wide. The canal roads are generally unsurfaced but made motorable by using compacted and dressed earth material. If the canal road is to be used for other purposes as well, it should be metalled and surfaced.

If a road is constructed on the ground along a banked canal, the canal is not visible while driving along the road. As such, the vehicle has to be stopped frequently at places to have a look at the canal. Thus, proper inspection of the canal is difficult. This problem can be avoided if the road and the bank are combined together, [Fig. 8.8(*b*)] and a 'dowla' of about 0.5 m height and about 0.5 m top width is provided for safety reasons. The level of the road is kept about 0.3 to 0.75 m above full supply level of the canal.

Schedule of Area Statistics and Channel Dimensions

The calculations for the design of an irrigation channel are usually carried out in tabular form. The table containing these calculated values is called the 'schedule of area statistics and channel dimensions'. The calculations start from the tail end and the design is usually carried out at every kilometre of the channel downstream of the head of the channel. Intermediate sections are designed only in special circumstances such as a large reduction in discharge due to an offtaking channel in between the adjacent two sections.

BORROW PITS, SPOIL BANKS, AND LAND WIDTH FOR IRRIGATION CHANNEL

Borrow Pits and Spoil Banks

Although it is advisable to keep the channel in balanced earth work, it is generally not possible to do so. If the amount of earth required for filling is more than the amount of excavated earth, then the excess requirement of filling is met by digging from suitably selected areas known as borrow pits, Fig. 8.11 (a).

If unavoidable, borrow pits should be made in the bed or berms of the channel. These pits will silt up after sometime when water has flowed in the canal. The depth of the borrow pits is kept less than 1 m and the width is limited to half the bed width. The borrow pits are located centrally in the channel bed and are spaced such that the distance between adjacent borrow pits is at least half the length of the borrow pits. Borrow pits may be similarly located in wide berms of a channel.

If the material from internal borrow pits is not sufficient to meet the requirement then extra material is taken from external borrow pits which should be about 5 to 10 m away from the toe of the canal bank. These should not be deeper than 0.3 m and should always be connected to a drain.

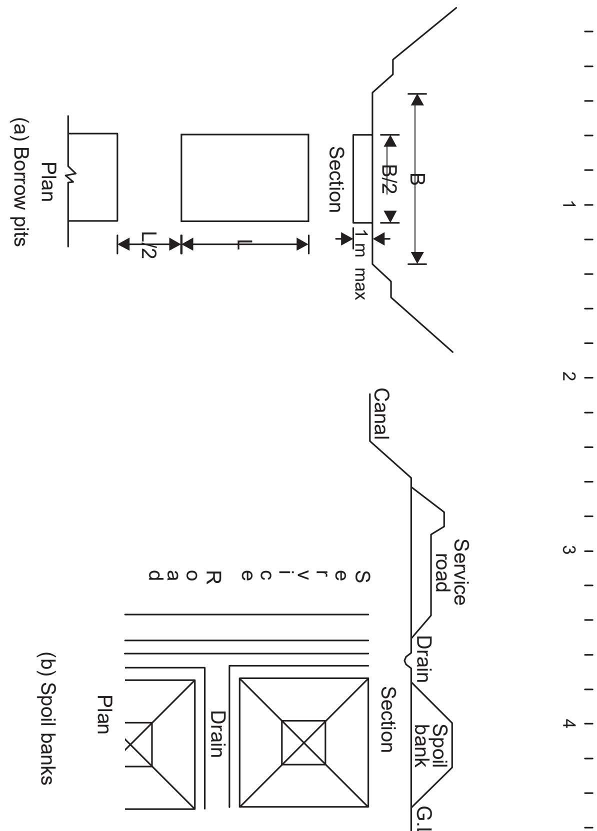


Fig. 8.11 Borrow pits and spoil banks

On the other hand, if the excavated earth exceeds the requirement of earth material for the construction of banks and service roads, the excess earth has to be suitably disposed of. If the excess earth is not much, it can be used to widen or raise the canal banks. If the quantity of excess earth is much larger, then it is utilised to fill up local depressions in the area or deposited in spoil banks, Fig. 8.11 (*b*), on one or both sides of the channel. The section of spoil banks depends on the cost of the land and labour. These should be provided with good cross slopes on all sides for ensuring proper drainage. The spoil banks should be discontinuous to allow cross drainage between them.

Land Width

Land width required for the construction of a channel includes the permanent and temporary land. The permanent land extends a little beyond the outer toe of the canal banks (or thedrain, if provided) on either side. If trees are to be planted along the canals, then the land acquired for this purpose should also be included in the permanent land. Planting of trees adjacent to good culturable land should be avoided as the shade of the trees would affect the plants.

In addition to the permanent land, some land along the channel is required during construction for the storage of materials and equipment, and other purposes related to the construction work. This temporary land is returned to the concerned owners after use withdue compensation.

2.4 Concept of Bio-Engineering for Watershed management

Green Infrastructure and Bio Engineering

Green infrastructure is a broad term used to describe measures where nature-based solutions form part of the infrastructure. Naumann et al. (2011) provide the following classification: “Green infrastructure is the network of natural and semi-natural areas, features and green spaces in rural and urban, and terrestrial, freshwater, coastal and marine areas, which together enhance ecosystem health and resilience, contribute to biodiversity conservation and benefit human populations through the maintenance and enhancement of ecosystem services. Green infrastructure can be strengthened through strategic and co-ordinated initiatives that focus on maintaining, restoring, improving and connecting existing areas and features as well as creating new areas and features."

Bioengineering is a subset of green infrastructure that uses vegetation to serve an engineering function. The most common uses of bioengineering include soil surface protection against erosion, soil stabilization, and improved drainage functions. While vegetation is widely used for landscaping, the focus of this brief is its use in different forms of infrastructure.

Bioengineering uses plants to protect soil surfaces and stream banks, and to strengthen shallow soil. It can control erosion and prevent or stabilize shallow slope movements where the depth to failure is no more than 0.5 meter (m). If the depth to the sliding surface of a slope failure is greater than 0.5 m, then bioengineering should only be applied in conjunction with other slope stabilization techniques, typically, retaining walls. Bioengineering techniques often provide the most cost-effective methods of surface protection for soil slopes, which is achieved through a surface cover of vegetation that armors the surface against erosion.

But soil protection is not bioengineering's sole application. Different types of plants and planting materials produce different rooting patterns, which can bind or anchor the surface layer of soil and increase its resistance to deformation. This prevents, or at least reduces, the incidence of deeper failures, such as large gullies resulting from the formation of shallow rills. Aboveground, the stems and leaves of plants can slow moving water and trap materials that are being carried by water or gravity. These features account for the range of engineering functions

Financial Implications

The cost and benefits of bioengineering are difficult to quantify because of the range of variables involved, and a lack of reliable input cost and monetized output benefit data. For example, slope-derived rock debris on hill roads is one of the largest contributors to road maintenance costs as it blocks drains and damages pavements, but there are no known calculations of the reduction in maintenance costs as a result of applying bioengineering techniques to prevent the debris from falling on the road. Similarly, the cost of losses in agricultural productivity due to erosion can be debilitating but demonstrating this at the farm level is difficult. In most cases, the attribution of benefits is hard to achieve without a formal trial with a control (no treatment) element.

The cost of construction using solely bioengineering measures was significantly low compared to the cost of conventional techniques for slope protection using concrete tiles (International Centre for Environmental Management 2017a). At one site in northern Viet Nam, the cost of bioengineering techniques ranged from 9.5% to 22.8% of their conventional alternatives. This series of trials found that the huge savings came from the protection of the slopes of riverbanks above hard-toe protection works. The International Centre for Environmental Management (2017a) also emphasized the inclusiveness and wage benefits from bioengineering works, which in the trials included a large proportion of women and ethnic minorities in the labor force.

1) Purpose

Table 1: Bioengineering Functions

Engineering Functions	Requirem
Catch eroding material moving down a slope, as a result of gravity alone or with the aid of water. The stems of the vegetation perform this function.	• Strong, numerous, and fle • Ability to recover from da
Armor slopes against surface erosion from both runoff and rain splash. This requires a continuous cover of low vegetation; plants with high canopies alone do not armor the slope.	• Dense vegetation cover • Low canopy • Small leaves
Reinforce the soil by providing a network of roots that increases the soil's resistance to shear. Reinforcement depends on the form of the roots and the nature of the soil.	• Plants with extensive root • Many strong, fibrous roots
Anchor surface material by extending roots through potential failure planes into firmer strata below. If the potential failure is deeper than 0.5 meter, anchoring can be achieved only by large woody plants with big vertical tap roots.	• Plants with deep roots • Strong, long, vertically orie
Support the soil mass by buttressing and arching. Large, heavy vegetation, such as trees, at the base of a slope can provide support in the form of buttresses; or on a micro scale, clumps of grass can buttress small amounts of soil above them. Across the slope, a lateral effect is created in the form of arching: this is where the soil between buttresses is supported from the sides by compression.	• Extensive, deep, and wide • Many strong, fibrous roots
Reduce the velocity of water or wind movement across the surface of the soil. This is done by the stems of vegetation offering resistance that retards the flow of water or air.	• Strong, numerous, and fle • Many strong, fibrous roots
Drain excess water from slopes. The planting configuration of the vegetation can	• Plants small enough to be

2) DesignConsideration

Generally, unstable bare slopes are unsuitable for vegetation due to frequent surface failures. There is little possibility of achieving success in developing vegetative cover on unstable slopes without supporting measures. Therefore, vegetation on the slope should be carried out when the slope is stabilized by itself or through implementation of counter measures.

For selection of vegetative countermeasures, species, and detail design considerations forimplementation and maintenance of bio-engineering works reference should be made to"RoadsideBio-engineeringSiteHandBook"and"ReferenceManual", and other relevant publications of DOR.

Water Management

Water management in both the cut and fill slopes is important to protect the slopes fromerosion and shallow depth instabilities due tosurface water and consequent increase inpore water pressure. Ingeneral, water management inslopes consists of surface and subsurface drainages that are capable to take away the water to the natural drainage system safely and as quick as possible. Studies regarding the rainfall, topography, catchment area, ground surface conditions, soil parameters, ground water conditions and existing natural and artificial drainage system should be carried out and assessed to determine the required drainage discharge. Combination of both the surface and sub surface drains could be effectively used to manage the surface and ground water conditions. Water management, being a quick and effective stabilizing measure on landslides and unstable areas, shall be considered as primary control measure. In case ofdistinctly visible cracks in slope, water infiltration in the ground is to be prevented bysealing cracks using clay or cement, and or polyethylene sheet.

Surface Drainage

Surface run off water from springs and rainfall should be prevented from infiltrating the slopes and/or landslide is at avoiding increase in pore water pressure. Incase of landslides, which are closely related to short-term rainfall, surface drainage works should be immediately executed without loosing time for the results from detail stability analysis. Surface drainage system comprises so catchdrain, berm drain, to e drain, drainage channels, and cascades. U-shapedgutter, reinforced concrete, corrugated halfpipe drain could be used to construct the drainage ditch. It should be checked that the surface water is properly collected in the ditch and once collected it should not infiltratethe slopes again. To improve the drainage function the drains are to be placed at lowestpoints oftheslopeswithproperliningandgradient.

1) Purpose

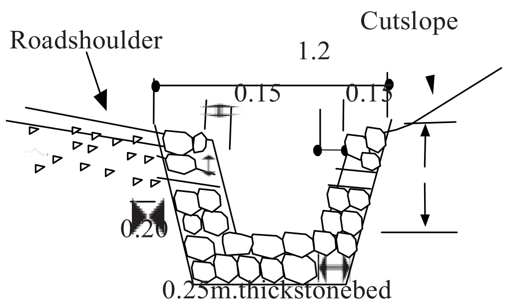
The main purpose of surface drainage system is collection of surface water due to rainfalland/or spring, and its safe discharge to the nearest natural drainage. Collection is donethrough catch drain andor numbers of collectors or tributary drains, and the collectedwater is quickly discharged through drainage channel or maindrains.

2) DesignConsideration

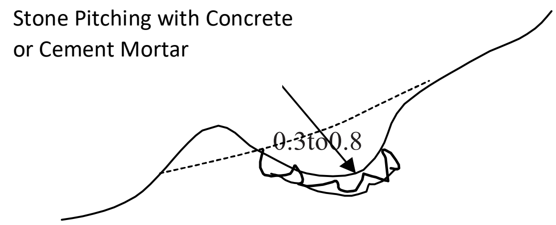
The design for the sizes of catch drain and the collector drains in the slope is based on theamount of surface runoff it has to cater. The amount of surface water could be estimatedbased on the intensity of rainfall, catchment area and characteristics of surface conditions.The drainage channel works, main drain and cascades (gabions/ masonry) are designed toremove the collected water out of the landslide zone as quickly as possible. The design ofsurface drainage system works are often combined with subsurface drains of up to 3 mdepthdependinguponthe necessityof drainagebelow theground.

Considering the importance of drainage structures, potential damages to the road pavement, retaining structures, and slope failures from concentrated runoff; it should be designed according to hydrological and hydraulic considerations. Hydrology such as frequency, intensity, and duration of rainfall, runoff peaks and their frequencies, groundwater table and its fluctuation are the most important concerns to the road engineers. Runoff could be estimated using the standard methods such as Rational Formula, US Soil Conservation Service Curve Number Method, and California Culvert Practice for Estimating Discharge. Flow in the drainage facilities such asside drain, catch drain, chutes, cascades and culverts that flowing partly full are designed according to principle of flow in open channel. The drainage channels are designed as uniform flow channelsconsidering the flow to be uniform in the constant cross section, roughness and gradient.Most widely used equation for the uniform flow is Manning's Equation with standard coefficient for different materials. Based on the estimation of design discharge from the vicinity and velocity of water in the drainage channel the discharge capacity of channel can be established.

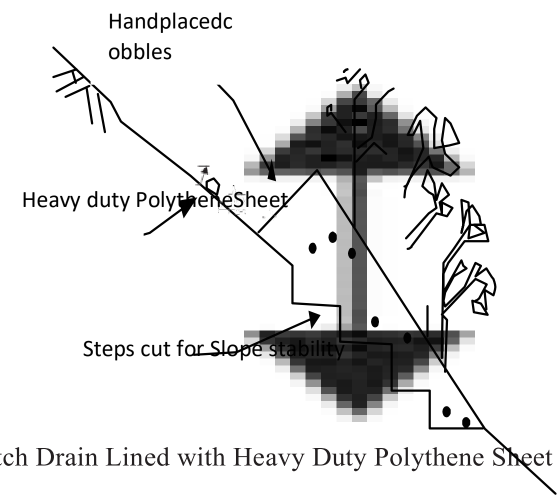
Collector or tributary drains are designed to collect surface water flow by installing corrugated half pipes or lined U-ditches along the slopes, which are then connected to adrainage channel or main drains. To prevent the infiltration of collected surface water thebed of drain is to be lined and or protected with polyethylene sheet. For cleaning and maintenance purpose suitable size of collection chambers should be designed and installed at the junction of collector or tributary drains and drainage channel, and also atpoints where the gradient and or direction of drain changes remarkably. In case of drainactive area of a landslide, drainage ditches should have the required strength and also beeasy for maintenance. The drainage channel or main drain along the steep slope should beconsidered for design of appropriate keying or anchoring structure in frequent intervals inorder to prevent sliding of drain. Interval of such keying structure basically depends up on the horizontal force created due to dead load of drain and the gradient of slope



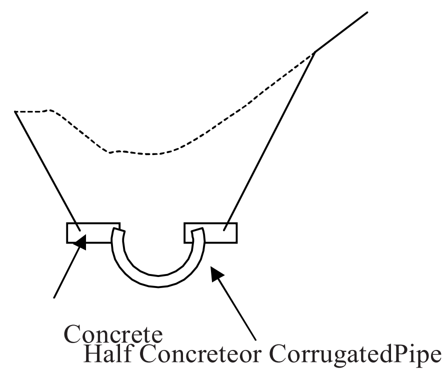
Trapezoidal Cement Masonry Side Drain



Stone Pitching Collector Channel



Catch Drain Lined with Heavy Duty Polythene Sheet



Half Pipe Drainage Channel

Figure3.6:Typical Surface Drains

3.1.1 HorizontalDrilledDrain

Ground water conditions in a slope affect the stability of slope.Ground water table usually rises in the rainy season causing saturation of soil mass and may result in the landslides Groundwater table can be at shallow or in depth. Shallow groundwater table (0to5meters below the ground surface) is mainly due to short-term rainfall, which frequently causes a localised shallow failure or toe failure in a large-scale unstable slope.Shallow groundwater is usually drained using the subsurface drains construction

When lowering of ground watertable is required from a depth higher than 5m, constructions of sub surface drains are almost impossible and risky due to excavation problem. In such case the deep groundwater table should be drained out by installation of horizontal drilled drains, drainage wells or drainage tunnels or their combination. Lowering of ground watertable through horizontal drilled drain sand drainage wells are one of the most effective methods of stabilizing landslides where the fluctuation of ground watertable is major cause for activating landslides.

1) Purpose

Horizontal drilled drain is used to lower both the shallow (to certain length) and deep ground watertable in a slope to help stabilization of the landslide.There duction in ground watertable in turn helpsto;

- decrease the pore water pressure with in soil mass,
- increase the shear strength along the slip surface,
- reduce the seepage force and erosion due to seepage,
- reduce the unit weight of soilmass by preventing it from soaking.

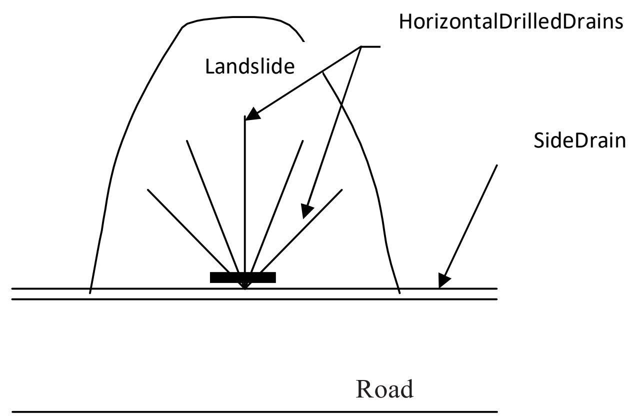
2) Design Consideration

Before designing the horizontal drilled drains, it is necessary to carryout geological andgeophysical investigations to find out whetherthe landslide orinstability iscaused by high seepage problems. It is also necessary to explore the depth and extent of ground watertable, its fluctuation and effect on movement of landslide. Permeable layers of soil mass, springs and aquifers are also to be checked for the necessity of horizontal drilled drains.

Based on the assessment and requirement of the slope conditions the horizontal drilled drains are designed with 20to50 meters in length. The diameter of bore holes vary from 50to100 millimeters depending upon availability of drilling equipment and are drilled at a gradient of 5 to 10 degrees. After drilling the drain hole suitable size of semi perforated HDP pipe (usually 50 mm dia.) is installed in the whole length of drainage hole. The perforation in the pipe is made on the upper half with 3 mm dia holes at the rate of 10 to15 mm distance and in zig-zag pattern. Wrapping of the pipe with geotextile with suitablefixtures, and support at the outlet end is required to make sure the pipe is not clogged anddisplaced quickly. At the drain outlet suitable support with cement masonry, concrete or gabion structure, and construction of open channel drain to discharge the collected groundwater to the nearest drainage system is required.

The numbers of horizontal drilled drain in the slope mainly depend on the wet area of slope that hasto bed rained.Usually the drains are made in fan shape and of shorter length. For drains with greater length there will be high risk of non-functioning due topossibilityof distortion due to movement.

For the ground water to flow in the horizontal drilled drain pipes the slope should not be of impervious materials or have lot of fines or clay. Typical sketches of horizontal drainareshown in the figure below.



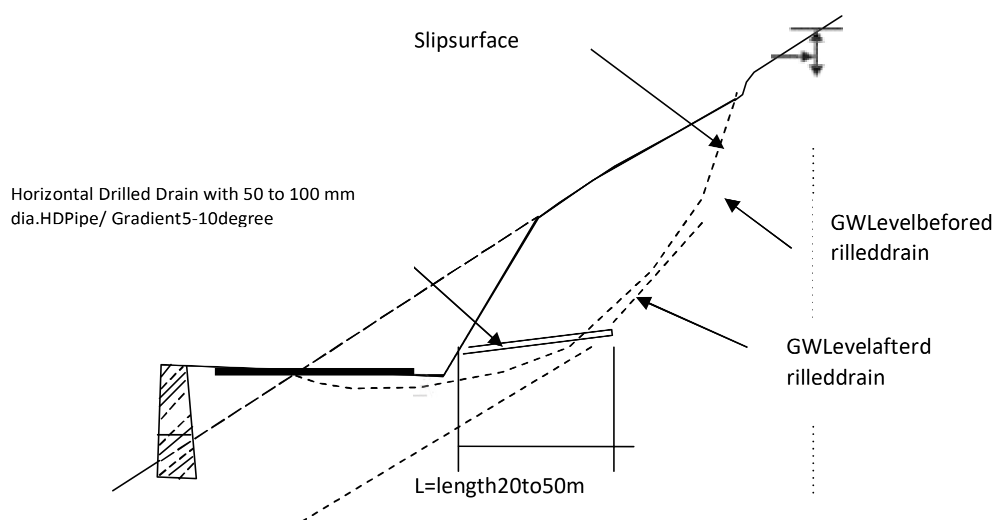


Figure3.7:Typical Plan and Section of Horizontal Drilled Drain

Figure3.8: Horizontal Drilled Drain

3.1.2 Sub-surfaceDrains

Sub-surface Drains (SSD) are effectively used to drain out the shallow ground water within 0 to 5 m below the ground surface. SSD collects the seepage water from surface runoff and avoids the increase in the ground watertable.

1) Purpose

The main purpose of the SSD construction is to collect and drain out groundwater of shallow depth of upto 3m depth. By removing such ground water it is possible to stabilize the shallow failures and also to reduce the groundwater table, hence reducing the risk of landslide due to pore water pressure.

2) DesignConsideration

For the design of SSD it is necessary to carryout geological and geophysical investigationto find outwhether the landslide or instability iscausedby high seepage problems.SSDare designed in the slope if the water seepage and sub-surface water are one of the maincauses of instabilities. The SSDs are placed quite similar to surface drains consisting ofcollector/tributary drain sand main drains.Inmany cases thesurfaceand subsurface drains are combined for effective drainage purpose.

The size and depth of collect or and main drains basically depends upon the area of slope to be covered by the SSD, rainfall intensity, and infiltration characteristics of the ground. Suitable sizes of perforated pipes with filter materials and or together with gabions/ drystone packing and geotextile materials are used to design the SSD.HDP pipes are recommended at bottom of SSD where amount of collected water is large. At bends,junctions of collector and main drain, and where the length of SSD are long it is recommended to make intermittent catch basin or manhole for clearing purpose. Care should be taken for prevention of infiltration of the collected water as it will be moredangerous to the slope stability. In many cases a polyethylene sheet are also used at thebottom ofthe SSDtostop theinfiltration.

When constructed along the slopes, the SSD may require construction of anchors or support structures at frequent intervals to prevent the drain from sliding. Design of suchanchor or support structure depends upon the slope angle, and size and construction ofSSD.

Figure 3.9 shows the network of subsurface drains consisting of tributary and main drains together with various drainage and retaining structure thatused to stabilize one of theactive and large scale landslide in Arniko Highway. Typical design of tributary and main drains are shown using gravel, drystone packing and gabions.

2.5 Integrated Water Resource Management (IWRM) and Role of Irrigation

As per the Global Water partnership (GWP) : IWRM is a process which promotes the coordinated development and management of water, land and related resources in order to maximize the resultant economic and social welfare in an equitable manner without compromising the sustainability of vital ecosystems.

AS per the USAID: IWRM is a participatory planning and implementation process, based on sound science, which brings together stakeholder to determine how to meet society's long term needs for water and coastal resources while maintaining essential ecological services and economic benefits. It helps to protect the world's environment, foster economic growth and sustainable agricultural development, and promote democratic participation in government, and improvement human health.

IWRM cannot be seen as a blueprint or product for good water management, but rather as a paradigm with a broad set of principles, tools, and guidelines that must be tailored to the specific context of a country, region, or river basin in order to implement an efficient and effective water resource management. A basic set of principles is outlined in Box.

IWRM principles

- Integrate water and environmental management.
- Follow a systems approach.
- Full participation by all stakeholders, including workers and the community.
- Attention to the social dimensions.
- Capacity building.
- Availability of information and the capacity to use it to anticipate developments.
- Full-cost pricing complemented by targeted subsidies.
- Central government support through the creation and maintenance of an enabling environment.
- Adoption of the best existing technologies and practices.
- Reliable and sustained financing.
- Equitable allocation of water resources.
- Recognition of water as an economic good.
- Strengthening the role of women in water management

Source: IWA/UNEP (2002)

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The IWRM Paradigm

The IWRM paradigm contains important key concepts of **integration, decentralization, participation, and sustainability** (Xie2006). Due to the holistic view of the IWRM paradigm, there is a necessity for the integrated management of horizontal sectors that use or affect water resources, e.g., water supply, sanitation, agricultural use, energy generation, industrial use, or environmental protection. In addition to horizontal integration, vertical integration is also required to coordinate efforts between local, regional, national, and international water user groups and institutions (Xie2006). The main aspects regarding natural system integration and human system integration are listed in detail in the chapter annex Sect.3.13.2(GWP2000).
Besides the necessity of integration, there is also need for decentralized decision- making and responsibility at the lowest effective management level, to increase awareness for local and regional problems. Hence, IWRM seeks to strike a balance between top-down and bottom-up management. IWRM also wants to strengthen community-based organizations and water user associations.
The consideration of sustainability, as a main part of IWRM, is not only restricted to ecological sustainability for protecting the natural system, but it also covers aspects of financial and economic sustainability. This means, for instance, that resource allocation decisions have to be based on the economic value of water. Therefore, water must be priced at its full costs (Xie2006).The three key policy goals of IWRM are Equity, Ecological integrity and Efficiency, which are known as the three'E's (Postel1992):

Equity: Water is a basic need and hence there is the basic right for everybody to have access to water of adequate quantity and quality.

Ecological integrity: Water in sufficient quantities with sufficient quality should persist in the environment. Water should be used in a sustainable way, so that the future generation will be able to use it in a similar way as the present generation.

Efficiency: Water must be used with maximum possible efficiency, because of its finite and vulnerable nature. Cost recovery of the water service should be attained. Water should be priced according to its economic value.

Approaches to IWRM

The Integrated Water Resource Management (IWRM) approach goes back to the establishment of the Tennessee Valley Authority (TVA) in the year 1933, which integrated the functions of navigation, flood control and power production (Biswas2004). Further issues, such as erosion control, recreation and public health, were also addressed by the TVA (Mitchell1990). The Secretary-General of the United Nations Organization (UNO) addressed the topic of IWRM in 1957. The UNO's understanding of integration refers to supporting services needed to develop irrigated agriculture, but the coordination of different water-related functions was not part of this IWRM concept. This deficit was remedied at the Water Conference in MardelPlatain1977where the necessity of coordination within the water sector was explicitly addressed. However, issues associated with high water demand and negative environmental impact so irrigated agriculture were not approached sufficiently (Snellen and Schrevel2004).

At the beginning of the1990s, there were some observable shortcomings in traditional water management, like quality issues, overexploitation, ecosystem degradation or social concerns. Water problems also had become multidimensional, multi- sectoral, and multiregional and filled with multi interests, multi-agendas, andmulti causes (Biswas2004).
To overcome these issues, four important guiding principles were determined during the International Conference on Environment and Water in Dublin in the year 1992 (Xie2006). These principles (ecological, institutional, gender,econ n nnnnnnnnnnnnnnnnnnnomic)becamewellknownasthe"Dublin-Principles",whicharestated in the annex of this chapter.
The Dublin Guiding Principles represented an important input for the Agenda 21,which was agreed upon the United Nations Conference on Environment and Development in Rio de Janeiro in 1992. Chapter 18 emphasized the need for an integresources by connecting different water ser- vices and providing good governance, appropriate infrastructure, and sustainable financing.

The present understanding of IWRM with its holistic approach is strongly based on the Dublin-Principles as well as on the Agenda21 document. There are many definitions of IWRM, for instance, in the Agenda21. A well-cited definition of IWRM is the one made by GWP (2000):

The Dublin Principles

Four important guiding principles were determined during the International Conference on Environment and Water in Dublin in the year 1992 with over 500 participants representing 100 countries and 80 international and nongovernmental organizations (Xie 2006). These principles are:

- **Principle No. 1 ("Ecological"):** Freshwater is a finite and vulnerable resource, essential to sustain life, development, and the environment. Since water sustains both life and livelihoods, effective management of water resources demands a holistic approach, linking social and economic development with the protection of natural ecosystems. Effective management links land and water use across the whole of a catchment area or groundwater aquifer.
- **Principle No. 2 ("Institutional"):** Water development and management should be based on a participatory approach, involving users, planners, and policymakers at all levels. The participatory approach involves raising awareness of the importance of water among policy-makers and the general public. It means that decisions are taken at the lowest appropriate level, with full public consultation and involvement of users in the planning and implementation of water projects.
- **Principle No. 3 ("Gender"):** Women play a central part in the provision, management, and safeguarding of water. This pivotal role of women as providers and users of water and guardians of the living environment has seldom been reflected in institutional arrangements for the development and management of water resources. Acceptance and implementation of this principle require positive policies to address women's specific needs and to equip and empower women to participate at all levels in water resources programs, including decision-making and implementation, in ways defined by them.
- **Principle No. 4 ("Economic**"): Water has an economic value in all its competing uses and should be recognized as an economic good. Within this principle, it is vital to recognize first the basic right of all human beings to have access to clean water and sanitation at an affordable price. Past failure to recognize the economic value of water has led to wasteful and environmentally damaging uses of the resource. Managing water as an economic good is an important way of achieving efficient and equitable use, and of encouraging conservation and protection of water resources.

Integration in IWRM

It is important to bridge components of the natural systems, like availability and quality of resources, as well as characteristics of human systems, which are fundamentally determined by resource use, waste production, and resource pollution. The main aspects regarding natural system integration and human system integration are listed in detail below (GWP 2000).

Natural system integration

- Integration of freshwater management and coastal zone management: Requirements of coastal zones have to be considered in upstream freshwater management
- Integration of land and water management: Land use influences the distribution and quality of water. Furthermore, water is a key determinant of the character of ecosystems.
- Distinction between "green water" and "blue water": Water that is directly used for biomass production and "lost" in evaporation is termed "green water", while "blue water" is the flowing water in surface and subsurface water bodies.
- Integration of surface water and groundwater management: An infiltration of water from groundwater bodies to surface water bodies and vice versa can occur.
- Integration of quantity and quality in water resources management: Aspects of generating, abating, and disposing of waste products have to be addressed.
- Integration of upstream and downstream water-related interests: Conflicts, interests, and trade-offs between upstream and downstream stakeholders using water resources have to be identified and balanced out

Human system integration

- Mainstreaming of water resources: The analysis of human activities have to involve the understanding of natural systems, its capacity, vulnerability, and limits.
- Cross-sectoral integration in national policy development: Water policy must be integrated with economic policy. The economic and social policy needs to take into account water resource implications.
- Macroeconomic effects of water developments: Water resource projects can have macroeconomic impacts (e.g., employment).
- Basic principles for integrated policy-making: Assess macroeconomic conditions of effects before realizing investment; weight expected (external) costs with (external) benefits of a policy; awareness of trade-offs in short-term and long-term
- Influencing economic sector decisions: Decisions impact water demands, availability, and quality.
- Integration of all stakeholders in the planning and decision process: Involvement of the stakeholders in the management and planning of water resources to deal with conflicting interests between stakeholders.
- Integrating water and wastewater management: Water is a reusable

resource, hence wastewater flows can be a useful additional resource.

Implementation of IWRM

Based on the GWP, the three main pillars for implementing IWRM in practice are an enabling environment, institutional roles, and management instruments (GWP (2004)):

The enabling environment

- Policies—setting goals for water use, protection, and conservation.
- Legislative framework—the rules to follow to achieve policies and goals.
- Financing and incentive structures—allocating financial resources to meet water needs.

Institutional roles

- Creating an organizational framework—forms and functions.
- Institutional capacity building—developing human resources.

Management instruments

- Water resources assessment—understanding resources and needs.
- Plans for IWRM—combining development options, resource use, and human interaction.
- Demand management—using water more efficiently.
- Social change instruments—encouraging a water-oriented civil society.
- Conflict resolution—managing disputes, ensuring sharing of water.
- Regulatory instruments—allocation and water use limits.
- Economic instruments—using value and prices for efficiency and equity.
- Information management and exchange—improving knowledge for better water management.

Some important points of Integrated Water Resources Management

- IWRM should be applied at catchment level. The catchment is the smallest complete hydrological unit of analysis and management. Integrated catchment management (ICM), therefore, becomes the practical operating approach. Although this approach is obviously sound and finds wide acceptance, too narrow an interpretation should be avoided.
- It is critical to integrate water and environmental management. This principle is widely and strongly supported. IWRM can be strengthened through the integration of Environmental Impact Assessments (EIA's), water resources modeling and land use planning. It should also be understood that a catchment or watershed approach implies that water should be managed alongside the management of codependent natural resources, namely soil, forests, air and biota.
- A systems approach. A true systems approach recognizes the individual components as well as the linkages between them, and that a disturbance at one point in the system will be translated to other parts of the system.

Sometimes the effect on another part of the system may be indirect, and may be damped out due to natural resilience and disturbance. Sometimes the effect will be direct, significant and may increase in degree as it moves through the system. While systems analysis is appropriate, analyses and models that are too complex to be translated into useful knowledge should be avoided.

- Full participation by all stakeholders, including workers and the community. This will involve new institutional arrangements. There must be a high level of autonomy, but this must at the same time be associated with transparency and accountability for all decisions. In this context Vision 21 states: A The real breakthrough came when the agencies all recognized that the most effective action came from the energy of people themselves. Care should be taken to ensure that those participating in any catchment management structure do indeed represent a designated group or sector of society. It is also important to ensure that representatives provide feedback to the constituencies they represent IWRM seeks to combine interests, priorities and disciplines as a multi-stakeholder planning and management process for natural resources within the catchment ecosystem, centered on water. Driven bottom-up by local needs and priorities, and top-down by regulatory responsibilities, it must be adaptive, evolving dynamically with changing conditions.
- Attention to social dimensions. This requires attention to, amongst other things, the use of social impact assessments, workplace indicators and other tools to ensure that the social dimension of a sustainable water policy is implemented. This will include the promotion of equitable access, enhanced role of women, and the employment and income implications of change.
- Capacity building. At many levels in the process Even at the governmental level - stakeholders lack the necessary knowledge and skills for full application of IWRM. Community stakeholders may not be familiar with the concept of water resource management, catchment management, corporate governance, and their role in these. Many, even in developed countries, do not even know what a catchment or watershed is. The water stakeholders must, therefore, collaborate in designing and implementing strategic elements of capacity building as part of the evolving IWRM process. Capacity building categories include education and awareness raising about water; information resources for policy making; regulations and compliance; basic infrastructure; and market stability. Early and ongoing stakeholder collaboration and communication in capacity building is also important from the point viewpoint of “leveling the playing field in anticipation of disputes that may arise. Filling strategic skills/capacity gaps supports IWRM, facilitates dispute resolution, and builds practical understanding of the scope of sustainable natural resource development challenges and opportunities.
- Availability of information and the capacity to use it to make policy and predict responses. This implies, firstly, sufficient information on

hydrological, bio-physical, economic, social and environmental characteristics of a catchment to allow informed policy choices to be made; and secondly, some ability to predict the most important responses of the catchment system to factors such as effluent discharges, diffuse pollution, changes in agricultural or other land use practices and the building of water retaining structures. The latter hinges on the adequacy of scientific models: Models should be as complex as the problem requires and no more so. It is recognized that predicting ecosystem response to perturbation with reasonable confidence is severely taxing current scientific capabilities, stimulating ongoing research.

- ❖ Full-cost pricing complemented by targeted subsidies. This principle was strongly urged by the World Water Council at The Hague, the rationale being that users do not value water provided free or almost free and have no incentives to conserve water. Wide support for this principle was engendered, but also significant opposition from those who felt that the interests of the poor might not be sufficiently protected, even under an associated subsidy system, however well designed. Opposing views held that full-cost pricing, when applied in its narrowest sense, offends the principle that water is a public good, a human right, and not simply an economic good. Reiterating: The economic sustainability of water and sanitation services depends largely and appropriately on the recovery of costs through user fees or tariffs that are equitably assigned based on ability-to-pay. Under-served or un served, marginalized users in many places already pay high financial costs of not having safe piped water, for example, because they are forced to pay for water trucked-in by suppliers. This water may be of dubious quality yet is expensive.
- ❖ Central government support through the creation and maintenance of an enabling environment. The role of central government in ICM should be one of leadership, aimed at facilitating and coordinating the development and transfer of skills, and assisting with the provision of technical advice and financial support, to local groups an individual. Where specific areas of responsibility fall outside the mandate of a single government department, appropriate institutional arrangements are required to ensure effective inter-departmental collaboration. Effective IWRM is a top-down meets bottom-up process.
  - Adoption of the best existing technologies and practices. This includes management instruments. Professional associations like IWA are primary sources of knowledge on BMPs (best management practices), and BAATs (best appropriate affordable technologies). Multi-stakeholder, consensus-oriented forums for IWRM should avoid lowest-common-denominator solutions through adherence to BMPs and BAATs that are adaptive to local needs.
  - Reliable and sustained financing. In order to ensure successful implementation of IWRM approaches, there should be a clear and long-term commitment from government to provide financial and human resources support. This is complemented by income from a healthy

water and sanitation market, especially when local providers of goods and services that support the water sector are active players, and when there is active reinvestment in the sector.

- Equitable allocation of water resources. This implies improved decision-making, which is technically and scientifically informed, and can facilitate the resolution of conflicts over contentious issues. There are existing tools (e.g. multi-criteria analysis) to help decision-making in terms of balancing social, ecological and economic considerations. These should be tested and applied.
- The recognition of water as an economic good. The recognition of water as an economic good is central to achieving equitable allocation and sustainable usage. Water allocations should be optimized by benefit and cost, and aim to maximize water benefits to society per unit cost. For example, low value uses could be reallocated to higher value uses such as basic drinking water supplies, if water quality permits. Similarly, lower quality water can be allocated to agricultural or industrial use.
- Strengthening the role of women in water management. A review by the World Bank of 121 water projects showed that ensuring women's participation in decision-making positively affects both project quality and sustainability.

IWRM in Nepali Context

In Nepal, opportunities for economic development and people's livelihoods depend on natural resources. Water resources are regarded as the key strategic natural resource that can be the catalyst for the country's overall development and economic growth. Contrarily, the country's natural resource bases have been undergoing rapid degradation. The links between poverty, financial incentives, institutional weaknesses and degradation of water, land and forest resources are distinct and visible. Degradation of the natural resources would mean diminishing the scope of economic development. It is also established that the management of a country's natural resources demands reforms in policy, institutions and governance and the people's practices alongside investments in physical infrastructures and inputs of technology. This justifies the relevance of an "integrated" approach to managing natural resources.

Policy provisions and legislation

As Nepal has three tiers of government, the development and management of water resources falls under the jurisdiction of all three, depending on the size of the project. Besides the deep-rooted indigenous customary laws, many statutory regulations have been promulgated and amended in the country's history. Despite all this, the government was operating without any appropriate or coherent policy until the 1990s. A paradigm shift was made by the Water Resources Act 1992 and Water Resources Regulations 1993 which supported the participation of users in water development projects. Separate Electricity Act and Regulations 1993 were enacted to specific legislation for the power sector where the main thrust is hydropower development with the promotion of private sector participation. However, all these legislative measures focused on the sectoral development of water projects where the fragmented approach of sharing a particular water source between various sectors like municipal, irrigation, hydropower and others gave rise to the possibility of conflicts.

For the first time in 2002, the government worked out the Water Resources Strategy (2002-27) as a policy and strategy document for water management. The National Water Plan 2005 was brought out with detailed plans and programmes alongside the estimated costs to support the strategy. River Basin Master Plans for all major river basins in the country are at different stages of development. The Irrigation Master Plan 2019, Irrigation Policy 2013 and National Water Resource Policy 2020 are among the significant policies and plans brought out by the government. The guiding principles of all these documents reflect the common agenda that the development of the country's water resources shall be managed holistically and systematically, relying on the principles of IWRM.

Further, it is stated that water utilisation shall be sustainable while ensuring the conservation of natural resources and protecting the environment. As far as the transboundary river basins are concerned, it is foreseen that sharing of water resource benefits among co-riparian countries shall be the essential feature of water sector management. This all shows that Nepal is well set for adopting the IWRM concept in its water resources management concerning policy provisions.

Missing links

Despite all these policy provisions, hardly any water development project has been implemented following the IWRM concept. While it is essential to have policies and plans, one significant shortcoming in the past has been lack of an effective implementation strategy that has limited the outcome of Nepal's development endeavours, including water resource management. Policies as such are not legal tools. Legislative measures supporting the policy provisions are essential. For a long time, a bill to amend the current Water Resources Act 1992 (which does not talk about IWRM) is still in the making. There is no legislative provision supporting the adoption of integrated planning and implementation of water projects. IWRM is about allocating water efficiently and equitably between various competing water uses in a river basin, and the distribution of resources in itself requires strong political commitment. In the absence of legal provisions for its implementation, it is not mandatory for the agencies responsible for planning and executing the water projects to follow the principles of IWRM, which asks for a paradigm shift in the age-old conventional sector-specific planning approach.

Two other significant pillars for IWRM planning and implementation are establishing, reorganising and activating an appropriate institutional set-up equipped with sufficient and well-trained interdisciplinary personnel, and providing adequate financial resources and the essential provision of management instruments. A mechanism for the participation of stakeholders needs to be established, and the government as a whole should facilitate the process. This has been foreseen in the Water Resource Strategy. Strengthening the Water and Energy Commission Secretariat as a central planning and coordination agency of the government has also been sought in the 2005 National Water Plan. On top of this, focus must be put on the promotion of the private sector and non-governmental organisations to support the process of IWRM implementation by ensuring transparency and effective stakeholder participation.

Plans to establish a knowledge-based information system at the Water and Energy Commission Secretariat and the River Basin Offices in the three major river systems—Koshi, Narayani and Karnali—have remained long overdue. Two decades have passed since the government approved the water strategy, but these provisions are yet to be realised. The planning and implementation of water development undertakings are still being pursued following the traditional sectoral fragmented approach. Not considering a river basin as a single planning unit for water development has begun to create conflicts between local level stakeholders and the government in some planned programmes, including the proposed Kaligandaki-Tinau Inter basin Water Diversion Project.

It is time to bridge these gaps and plan and execute water development programmes holistically, leading to accomplishing the goals set in the SDGs, to which all UN member countries, including Nepal, have expressed their commitment.

बहुउद्देश्यीय आयोजना विकासको आवश्यकता

नेपालमा उपलब्ध जलस्रोतलाई बहुआयामिक फाइदा हुने गरी बहुउद्देश्यीय आयोजनाको रुपमा विकास गरी देशको दिगो आर्थिक वृद्धि गर्नु वर्तमानको आवश्यकता रहेको छ को फाइदाहरुबहुउद्देश्यीय आयोजना |

- बहुउद्देश्यीयजलाशययुक्त र अन्तर जाल्धर जल स्थानान्तरण , आयोजनाहरुको विकसबाट सिंचाई, जलबिधुत, बढी नियन्त्रित, खानेपानी, आमोद, प्रमोद, पर्यटन तथा वातावरणीय फाइदा लिन सकिने हुनाले यस बाट उपलब्ध जलस्रोतको अधिकतम उपयोग गर्न सकिने साथै देशको दिगो आर्थिक वृद्धि गर्न सकिनेछ

- बहुउद्देश्यीय र जलाशययुक्त आयोजनाहरुको विकासबाट जलबिधुत विकास, कृषि उत्पादन वृद्धि गरी हाल विधुत तथा कृषिमा भएको आयातको असन्तुलन कम गर्न मद्दत पुर्याउने
- नेपालको Hydrology अनुसार जाल्स्रोत्को समय र स्थानको आधारमा हुने उपलब्धता र माग बिचको अन्तरलाई भेरियेशनलाई / सम्बोधन गर्ने उपयुक्त विकल्प हुने
- तराई भूमि उर्बर र कृषिको लागि उपयुक्त रहेकोले बहुउद्देश्यीय , जलाशययुक्त र अन्तर जलाधार जल स्थानान्तरण आयोजनाको नाहरुमा बार्हैं महिना पानी उपलब्ध हुने विकासबाट सिंचाई आयोज साथै सतह र भूमिगत जलस्रोतको संयोजनात्मक रुपमा दिगो उपयोगिताबाट कृषि उत्पादन तथा उत्पादकत्व वृद्धि गर्न
- तराई मधेश क्षेत्रमा भूमिगत जलस्रोतको उपयोगबाट भू पानीको सतह घट्न थालेकोले यस्ता आयोजनाबाट भूमिगत जलस्रोत रिचार्ज हुनुको साथै यस्ता आयोजनाको विकाशबाट दीर्घकालीन रुपमा दिगो जलस्रोतको व्यवस्थापन हुनुको साथै वातावरणीय फाइदा लिन
- जलवायु परिवर्तनको कारणबाट आउने समस्याहरु निराकरणको लागि बहुउद्देश्यीय जलाशययुक्तअन्तर जल स्थानान्तरण जस्ता , क्नेआयोजनाको विकास उपयुक्त विकल्प हुन् स

बहुउद्देश्यीय आयोजना विकास तथा संचालन गर्न गरिएको नीतिगत व्यबस्थाहरु

१. सिंचाई नीति २०७०

- जलस्रोतको अधिकतम उपयोग तथा विकास गर्न बहुउद्देश्यीय आयोजना, जलाशययुक्त आयोजना र अन्तर जलाधार स्थानान्तरण आयोजनाहरुको विकास गर्ने
- विगतमा विकास गरिएको सिंचाई संरचनाहरुबाट बर्षे भरी सिंचाई सुबिधा उपलब्ध गराउनको लागि संभाव्यताका आधारमा अन्तर जलाधार जल स्थानान्तरण हुने आयोजना, जलाशययुक्त आयोजनाको

निर्माण गर्ने साथै सतह र भूमिगत जलस्रोतको संयोजनात्मक उपयोग गर्ने

- सिंचाई बिकासमा वेदेशी तथा स्वदेशी निजी लगानीकर्तालाई आकर्षित गर्ने
- ठुला तथा बहुउद्देश्यीय आयोजना कार्यान्वयन गर्न आवश्यक स्रोत जुटाउन सिंचाई विभागले सर्व साधारण स्थानीय उपभोक्ता तथा बैंक एवं वित्तीय संस्थालाई कानुनी व्यवस्थाको आधारमा सेयर जारी गर्न सक्ने
- सिंचाई प्रणालीको निर्माण, संचालन र व्यवस्थापन गर्न निजी क्षेत्र, सहकारी र समुदायसंग सहकार्य साझेदारी गर्ने ,

२. जलस्रोत ऐन २०४९

- जलस्रोतको स्वामित्व नेपाल सरकारमा रहने व्यबस्था साथै बहुउद्देश्यीय आयोजनाको विकास गर्न निजी क्षेत्रको सहभागिता गराएमा वा सार्वजनिक निजी साझेदारीमा विकास गर्ने भएमा अनुमतिपत्र अनिवार्य रुपमा लिनुपर्ने
- जलस्रोत तथा सिंचाई विभाग वा यस अन्तर्गतको निकायबाट बहुउद्देश्यीय आयोजनाहरुको विकास गर्न सकिने
- नेपाल सरकारले कुनै स्वदेशी वा बिदेशी कम्पनीसंगठित संस्था वा , करार गरी /व्यक्ति संग प्रचलित कानुनको अधिनमा रही निश्चित शर्त जलस्रोतको विकास, उपयोग र सेवा विस्तार गर्न सकिने
- सरकारले विकास गरेको आयोजनाहरु उपभोक्ता संस्थालाई हस्तान्तरण गर्न सक्ने
- जलस्रोतको विकास गरी सेवा प्रदान गरे वापत सरकार वा निजी क्षेत्रले सेवा शुल्क लिन सक्ने

३. विधुत ऐन/२०४९नियमावली /२०५०

विधुतको सर्वेक्षणउत्पादन ,, प्रसारण वा वितरण गर्न व्यक्ति वा संगठित संस्थालाई अनुमतिपत्रको व्यवस्था गर्न सकिने र सो सम्बन्धि विभिन्न व्यवस्थाहरु समावेश गरेको

४. विकास समिति ऐन २०१३

- नेपाल सरकारले बहुउद्देश्यीय आयोजना, जलाशययुक्त आयोजना र अन्तर जलाधार स्थानान्तरण आयोजनाहरुको विकास गर्न उचित र आवश्यक ठानेमा विकास समिति गठन गर्न सक्ने व्यवस्था

५. सार्वाजनिक निजी साझेदारी नीति २०७२आदिमा समेत २०७५ऐन / बहुउद्देश्यीय आयोजना, जलाशययुक्त आयोजना र अन्तर जलाधार स्थानान्तरण आयोजनाहरुको विकास गर्ने निजी क्षेत्रलाई आकर्षित गर्ने उल्लेख

बहुउद्देश्यीय सिंचाई आयोजनाको बिकासका विधी एवं प्रक्रियाहरु

नेपाल सरकारले हालसम्म बहुउद्देश्यीय सिंचाई आयोजनाहरुको विकास पुर्णरुपमा सरकारको आफ्नो लगानी तथा दातृ निकायको ऋण सहयोगमा गर्ने गरेको छ | | ता लागू भए तिनै तहका सरकारहरु विच राजस्व बाँडफाँड गर्नुपर्ने हुन्छसंघिय यसले आगामी दिनमा राज्यले ठुला तथा बहुउद्देश्यीय सिंचाई आयोजनाहरु विकासको लागि पुँजी जुटाउन बैकल्पिक उपायहरु अबलम्बन गर्नुपर्ने आवश्यकता ति मध्ये नि | देखिन्छम्न मोडेलहरु हुन् सक्छ |

- नेपाल सरकारको आफ्नो लगानी
- वैदेशिक ऋण
- निजी क्षेत्रको लगानी
- EPSF)Engineeringprocurementconstructionfinancing(
- सार्वजनिक निजी साझेदारी
- विकास समिति कम्पनि मोडेल आदि

१. नेपाल सरकारको आफ्नो लगानी

यस्तो लगानीमा दाताको कुनै शर्तहरु बाध्यकारी नहुने साथै आफ्नो हिसाबले विकास गर्न सकिन्छ र वित्तीय हिसाबले आकर्षक नभएका आयोजनाहरु तर आर्थिक सामाजिक दृष्टिकोणले अत्यन्त आवश्यक हुन सक्ने आयोजनाहरु विकास गर्न सकिने

तथापी, लगानीको सिमितताको कारणले धेरै आयोजनाहरुको विकास एकै पटक गर्न नसकिने सथिया बजेट विनियोजन न्यून हुने हुनाले आयोजना सम्पन्न गर्न लामो समय लाग्ने र लागत वृद्धि समेत हुनसक्ने

२. वैदेशिक ऋण

दातृ निकायहरुबाट सहुलियत ऋणव्यापारिक ऋण अनुदान लिएर प्राथमिकता , प्राप्त जलस्रोतका ठुला बहुउद्देश्यीयआयोजनाहरु निर्माण गर्न सकिने तर तल्लो तटीय असार पर्ने जलस्रोतको क्षेत्रमा दातृ निकायले लगानी नगर्ने, दातृ निकायको अनावश्यक शर्तहरु रहने, आयोजनात्यरिमा धेरै समय सहित निर्माणमा अझै बढी समय लाग्ने, समाजिक वातावरणीय तथा आर्थिक विश्लेषण गर्न ,, तथा अनुपालन Complianceमा लामो समय र खर्च लाग्न सक्ने

३. निजी क्षेत्रको लगानीमा)VGFसहित(

४. EPCF

५. सार्वजनिक निजी साझेदारीमाBOT,DBOetc.

बहुउद्देश्यीयजलाशययुक्त आयोजनाहरु प्रतिफलका हिसाबले आकर्षक हुने हुँदा , बैदेशिक लगानी बाट निर्माण गर्न सकिने तर/निजी क्षेत्रको प्रत्यक्ष लगानीत्यसका लागि सरकारले जग्गा प्राप्ति, वातावरणीयबाढी नियत्रण ,, सिंचाई जस्ता बिषयबस्तुमा VGFसमेत आवश्यक नीतिगत र कानुनी प्राबधानहरुलाई समय सापेक्ष गर्नुपर्ने हुन् सक्छ।

६. कम्पनि मोडेल

कम्पनि ऐन अनुसार कम्पनि गठन गरी निजिक्षेत्र ,तिनै तहको सरकारको लगानी , सिंचाई विभाग, विधुत बिभाग को लगानी र स्वदेशीविदेशी बैंक बाट ऋण / आदि व्यवस्था गरी बहुउद्देश्यीय आयोजनाहरु निर्माणगर्ने सकिने

७. विकास समिति

बहुउद्देश्यीयजलाशययुक्त र अन्तर जलाधार जलस्थानान्तरण आयोजनाहरुको , संचालन र व्यवस्थापन विकास समिति गठन गरेर ज ,विकाससमा सरकारको पूर्ण लगाई तथा वैदेशिक दातृ निकायबाट ऋण लिई समेत गर्न सकिने

बहुउद्देश्यीय आयोजनाको संचालन तथा व्यवस्थापन विधि

१. सिंचाई संरचनाहरुको संचालन र व्यवस्थापन

आयोजनाको बाँध, हेडबोर्कसमूल नहर ,, शाखा नहरहरू मध्ये सिंचाई नीति अनुरुप ठुला सिंचाई आयोजनाहरुमा मूलनहर भन्दा तलका संरचनाहरु जलउपभोक्ता संस्थाले संचालन, मर्मत तथा सम्भार कार्य गर्न सकिने

२. जल विधुत गृहको संचालन र व्यवस्थापन विकल्पहरु मा जलस्रोत तथा सिंचाई विभाग आफैले वा बिधुत प्राधिकरणलाई हस्तान्तरण गरेर

३. लिज, संचालन तथा हस्तान्तरण

४. व्यवस्थापन करार

५. स्वामित्वकरण, संचालन र हस्तान्तरण

६. विकास समिति

७. क

म्पनि मोडेल आदि

अन्तर जलाधार जल स्थानान्तरणको अवधारणा

- बिगत ६ दशक देखि यसको थलानी
- अवधारणाको मूल उदेश्य जल अभाव हुने जालाधर क्षेत्रमा जलस्रोतको दिगो व्यवस्था गरी बर्षे भरि सिंचाई सुविधा पुर्याई कृषि उत्पादन, उत्पादकत्व बढाउदै गरिवी निवारण गर्दै समृद्ध नेपाल र आत्मनिर्भर मुलुक बनाउने जसले बाढी नियन्त्रण, प्रयार्प्त बिधुत उत्पादन, रोजगारी सिर्जना गर्ने, वातावरण संरक्षण समेत गर्ने
- हालसम्म एकल उदेश्य सिंचाई प्रणाली मार्फत सिंचाई को विकास भएको तर समयक्रम संग सम्पूर्ण सिंचित भूमिमा बर्षे भरि सिंचाई पुर्याउन बहु उदेशीय सिंचाई प्रणालीको विकास गर्नुपर्ने आवश्यकता पूर्तिको लागि सिंचाई नीतिको परिकल्पना गरे बमोजिम एक पछि अर्को अन्तर जालाधर जल स्थानान्तरणका आयोजना शुरु भएको

- सुनकोशी कमला, मरिनभेरी बबई जस्ता नदीहर ,ु मार्फत सिंचित गर्ने प्राथमिक अवधारणा तर विगतमा प्राविधिक जनशक्तिको कमि, लगानीको अभाव, technologyको समस्या आदिको कारण अगाडी नबढेको
- नेपाल जलस्रोत रणनीति २०५९, NWP२०६२ मा रणनीतिक रुपमा यस सम्बन्धि थप व्यवस्थाहरु गरेको र सोहीअनुसार प्रोजेक्टको inceptionदेखि बिस्तृत अध्ययनको थालनी हुदै गरेको
- जलस्रोतको एकिकृत व्यवस्थापनको सिद्दान्तलाइ सिंचाई नीति २०७० ले समेत उजागर गरेको
- तत् पश्चात् करिव ५० बर्ष अघि देखिको सपनालाई साकार पार्न बबई नदीमा snowfed नदी भेरीमा चुरे पहाड भित्र भित्रै सुरुंग बनाई पानी खसाल्ने कार्यको लागि सम्पूर्ण तयारीका साथ निर्माण कार्य अगाडी बढेको छ भन्ने सुन कोशी मरिन जस्ता आयोजना समेतको कार्य शुरु भएको साथै तमोर चिसांगसमेत कालिगण्डकी तिनाउ जस्ताको अध्ययन , भैरहेको
- यस्ता प्रणालीको बिकासमा आउन सक्ने जलस्रोत माथिको अधिकार , वातावरण सम्बधित आवश्यकता, लगानी र प्रतिफलपानीको , अग्राधिकार, बिभिन्न प्रयोजनको लागि जलको बांडफांडजस्ता , बिषयबस्तुमा ठोस रणनीतिको आवश्यकता पर्ने हुँदा सोहीअनुसार तयारी अगाडी बढाउनुपर्ने
- जलवायु परिवर्तनको कारण सहित विभिन्न कारणले सतह तथा भूमिगत जलस्रोतको मात्र घट्दै गई रहेको र उपयोग भने बढिरहेको सन्दर्भमा पानीको प्रयोगको प्राथमिकता, लाभदायिक उपयोगका शर्तहरु एवं पानी संरक्षणको बारेमा समेत थप स्पष्ट हुन् जरुरी

Water security in Nepal

The concept of water security has received increased attention in recent years in both policyand academic debates (Cook & Bakker, 2012; Salam, Shrestha, Pandey, & Anal, 2017). Nepalhas high annual rainfall, but still faces considerable challenges in ensuring water security. This is to a great extent the result of the high temporal and spatial variations in water availability as well as the lack of congruence between locations of water availability and water need (Water and Energy Commission Secretariat [WECS], 2005), but it is also related to governance issues. The challenges to water security in Nepal include ensuring improved access to safe drinking water and sanitation, providing sufficient water for irrigation, securing water for urban needs, and generating energy without compromising the water-dependent ecosystems.

Water security as a concept has evolved over time. In the 1990s, the concept was mainly human-centric (Cook & Bakker, 2012), but it is now becoming much broader. The GlobalWater Partnership (2000) has defined water security as 'access to adequate safe water at an affordable cost ensuring that the natural environment is protected and enhanced'; thus incorporating ecological dimensions. This approach has been further broadened by the increasing recognition that water resources management is complex and intertwined with the development and social sectors (Biswas, 2004; Falkenmark, 2001). More recently, UN-Water (2013) defined water security as 'the capacity of a population to safeguard sustainableaccess to adequate quantities of acceptable quality water for sustaining livelihoods, humanwell-being, and socio-economic development, for ensuring protection against water-bornepollution and water-related disasters, and for preserving ecosystems in a climate of peaceand political stability', thus balancing the needs for water management related to socio- economic development, health, disasters and ecosystems. Further, it is now recognized that water security cannot be treated as a stand-alone issue. Attention is turning to the water-energy-food (WEF) nexus: the complex interactions within the wider framework of water, food and energy which mean that the actions in each sector can have effects in the others,and that water security, energy security and food security are closely linked (FAO, 2014).Rasul (2014) emphasized the role of the Hindu Kush Himalaya (HKH) ecosystem services in sustaining all three in downstream areas, and the importance of the nexus in South Asia resulting from the high dependency of downstream communities on upstream ecosystem services. In this wider framework, investment in water infrastructure and securing water is equally important to bring economic productivity and growth, and water security can be considered as the longer-term payoff of higher economic growth and lower poverty (Asian Development Bank [ADB], 2016; Rasul, Neupane, Hussain, & Pasakhala, 2019).

Despite having an estimated 7000 m^3 per person per year of water resources (FAO, 2016),Nepal's water security is thought to be among the weakest in Asia and the Pacific (ADB, 2016). Improving water security requires better understanding not only of water supply and demand but also of the linkages within the WEF nexus, and of the aspects within it which can be addressed to achieve overall improvements in water security. Most water security studies in

Nepal have been context-specific. For example, Regmi (2007) looked into water security in the context of a farmer management irrigation system; there have been studies related to gender and poverty (Upadhyay, 2003) and agricultural insecurity (Dahal et al., 2015); and Biggs, Duncan, Atkinson, and Dash (2013) highlighted how poor governance has hindered Nepal in achieving water security. Some studies (e.g. Bradley & Bartram, 2013; Brown, Meeks, Ghile, Hunu, & Brown, 2013; Garrick & Hall, n.d.; Hanjra & Qureshi, 2010; Miyan, 2015; Thapa, Ishidaira, Pandey, Bhandari, & Shakya, 2018) have looked at water security as a small part of larger studies on geographical coverage and context.

This article assesses various aspects of water security in Nepal from the water-energy- agriculture (food) nexus perspective, following the framework shown in Figure 1. It looks at overall water availability and the challenges to and drivers of water insecurity, and then unravels the WEF nexus to reveal specific issues in selected water use sectors and other aspects related to water, and what development can be encouraged in other parts of the nexus that might improve water security and in turn contribute to the national economy. The article considers some specific measures to improve water security in different sectors and in particular provides some insight into the role of groundwater in agricultural water security and its relationship to energy provision, and the important role of governance in addressing water security using a holistic and integrated approach. It concludes by suggesting that a nexus-based approach is required for effective water management and governance, and that Integrated Water Resources Management (IWRM) and Integrated River Basin Management (IRBM) approaches need to be broadened to take into account the nexus between water, energy, and food production and security.

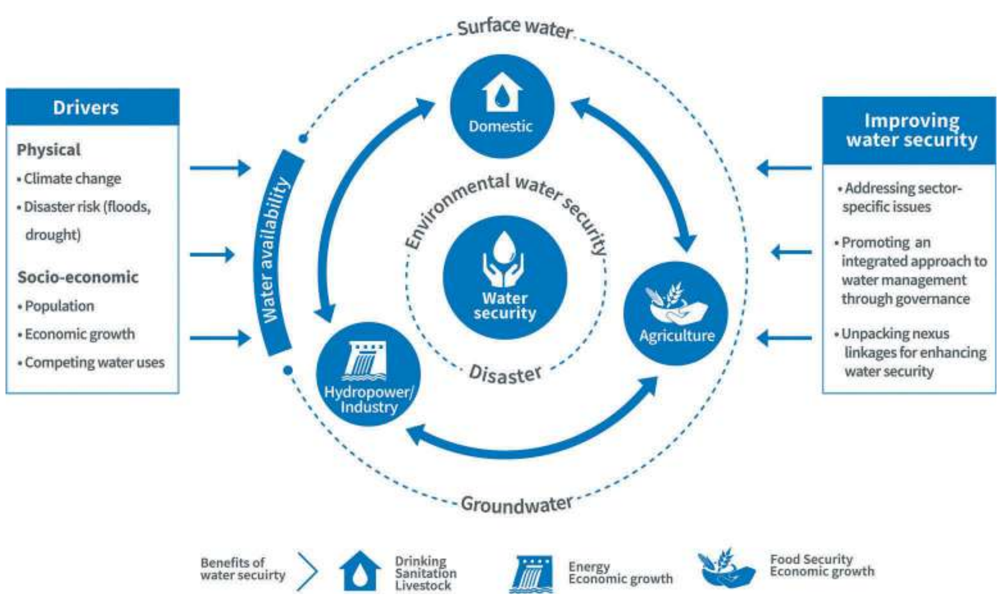


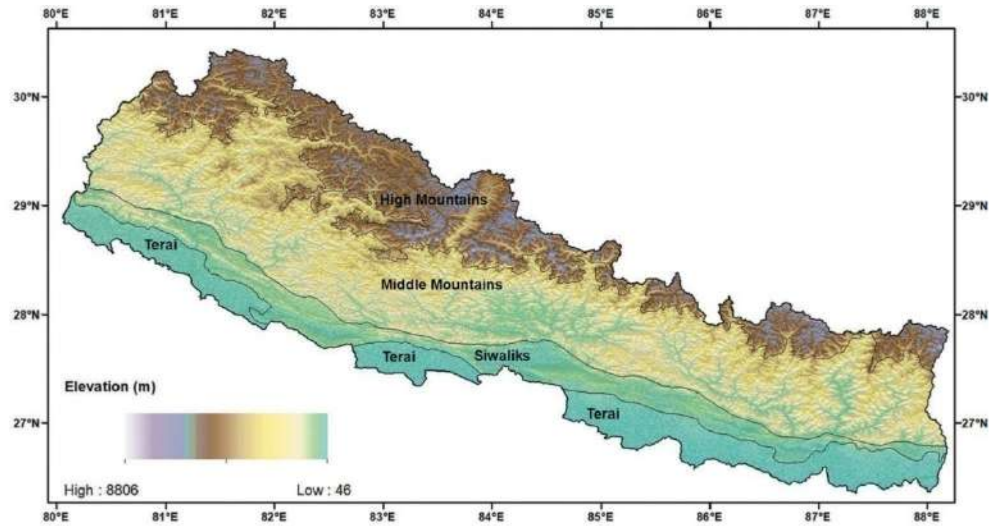
Figure 1. Conceptual framework used for analyzing water security.

Water availability

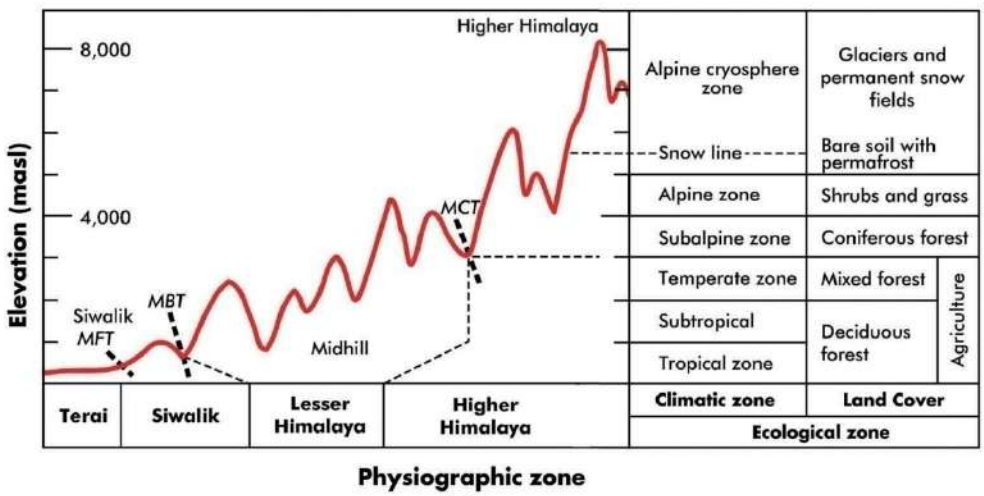
Nepal receives abundant precipitation annually, but it is distributed very unevenly as a result of both the monsoon climate (with marked temporal variation) and the extreme topography (leading to marked spatial variation). Water resources are available mainly in the form of surface water and groundwater, both of which have strong spatial and temporal variability.

Topography and climate

Nepal's extreme topography heavily influences the pattern of precipitation. The elevationranges from below 70 masl in the south to 8848 masl in the north, increasing rapidly within less than 200 km. As a result, Nepal has four main physiographic regions, with seven main climatic and ecological zones (Figure 2). The differences in climate and topography are reflected in differences in water availability and use, land use and land cover, population density and livelihood patterns. The Terai in the south is relatively flat, with a tropical to sub-tropical climate, good water resources and high net primary productivity, and thus high population density. The Terai is bounded by the low hills of the Siwaliks to the north, followed by the largest region of the Middle Mountains (Lesser Himalaya). The deep valleys and steep slopes encompass subtropical, temperate and subalpine zones and are dominated by anthropogenic activities, especially terraced agriculture and livestock farming, but with a low population density compared to the Terai. Hydropower production is mostly located in this region. The Higher Himalaya to the north is dominated by a sub-alpine to alpine climate, with rangelands, glaciers and snow peaks, fewer anthropogenic activities, and a low population density.



(a)



(b)

Figure 2. The Nepal Himalyas. (a) Physiographic zones. (b) Ecological zones along a typical south-north transect (source: Nepal, Flügel, Krause, Fink, & Fischer, 2017). (Notes: The Lesser Himalaya and Higher Himalaya are also known as the Middle Mountains and High Mountains, respectively. MFT = main frontal thrust; MBT = main boundary thrust; MCT = main central thrust; masl = metres above sea level.)

Precipitation in Nepal is dominated by the summer monsoon and the winter westerlies. Average annual precipitation (1971–2014) is 1450 mm (DHM, 2017), of which 80% falls in the monsoon season (June to September), and 20% during the other eight months (Figure A1 in the supplemental online resources). The spatial variation is also high, with average annual precipitation in different districts ranging from less than 300 mm in the northern mountains to more than 4000 mm in the eastern Middle Mountains (Figure A1). In the dry seasons, snow and glacier melt and groundwater sustain flow to the major rivers and provide the main source of water for different uses. Especially upstream, melt runoff in the pre-monsoon period (spring) supports irrigated agriculture along the river valleys.

Surface water availability

All the major river systems in Nepal are transboundary, originating in China and flowing through Nepal to join the Ganges in India (Figure A2). The total average annual runoff from all the river systems is estimated at 225 billion cubic metres (BCM), of which 172 BCM originates in Nepal (Table A1) (WECS, 2005). River flow is highly seasonable, with about 75% occurring in the monsoon season (June to September) (Figure A2). The high rainfall and river discharge lead to landslides, floods and flash floods in both mountain and Terai regions, but this is also a good time to plant major irrigated crops, such as paddy. Areas supported by year-round irrigation systems can grow more than one crop.

Groundwater availability

Groundwater is a reliable and flexible source of water for irrigation and other purposes, with many advantages compared to canal irrigation (Siebert et al., 2010). Groundwateris available in most parts of Nepal at varying depths and amounts. The southern Terai plains are in the groundwater saturated zone of the Indo-Gangetic Basin, whereas intra- mountain valleys such as the Kathmandu and Dang have isolated groundwater basins. The total volume of groundwater potentially available for extraction has been estimated by various authors using a variety of methods. Shrestha, Tripathi, and Laudari (2018) estimated annual renewable groundwater resources in the Terai at 8.8 BCM using measurements of seasonal fluctuations in the water table in shallow tube wells(2.5 m deep).

Despite this potential, groundwater use in Nepal remains relatively low. The AQUASTAT (2018) historical database on groundwater shows that it was almost nil in the 1980s, when groundwater extraction was already increasing in India, Bangladesh and Pakistan. Shrestha et al. (2018) estimated current groundwater extraction of 1.9 BCM/y in the Terai for irrigation, domestic and industrial use, compared to annual recharge of 8.8 BCM – an annual groundwater balance of +6.9 BCM/y (Table 1). This suggests a huge potential in renewable groundwater reserves.

Table 1. Groundwater balance in the Terai region of Nepal, in billion cubic metres

Annual groundwater recharge	8.8
Annual groundwater extraction	
Irrigation from shallow tube wells	1.16
Irrigation from deep tube wells	0.15
Domestic use	0.46
Industrial use	0.16
Total	1.93
Annual balance (recharge minus extraction)	6.9

Source: Shrestha et al. (2018).

Challenges to water security

Despite its large water potential, Nepal has very low water security, with many people unable to access sufficient water to meet their domestic, agricultural and industrial needs. There are many factors contributing to this. Some of the more important are outlined below. They are divided into specific challenges in different water use sectors and overall management, and drivers that are likely to challenge overall water securityin the future.

Challenges in specific water use sectors

Agricultural water security

Different factors hinder access to water in the hills, mountains and plains of Nepal. In the hills and mountains, the main problems are geophysical, with water mostly accessible in the valleys below the steep slopes, and rocky subsoil limiting the possibility of storage. Thus, rainfed agriculture is still the method of choice in most of this area. The plains areas of the Terai are mostly suitable for irrigation, but irrigation use remains low. A number of reasons can be identified:

- Most irrigation in the Terai is via canals that take water from small and medium-size rivers (originating in the Siwaliks and Mahabharat range) which dry up in winter and summer. Thus, the irrigation command area is much larger in the monsoon season than in summer and winter. Year-round irrigation coverage is low, and the area for growing winter and summer crops is relatively small.
- Energy constraints limit the ability of farmers to pump groundwater. Of the roughly 100,000 shallow tubewells in Nepal (most in the Terai), almost 80% use diesel pumps. Diesel is both expensive and dirty and puts pressure on foreign exchange. So far, the national grid has not reached all rural areas, and where it has reached, farmers have not always been able to access the subsidized farm power connection. And even where farm power is available, the electricity supply is intermittent and unreliable. This leads to a paradox of having plentiful groundwater at relatively shallow depths in most parts of the Terai, but low utilization of the resource due to high energy costs.
- Irrigation infrastructure in Nepal tends to be of a traditional type, which requires more labour and resources for repair and maintenance. Increasing erosion, landslides and sediment have further complicated repair and maintenance, making it more costly and reducing the command area for surface irrigation. Overall, irrigation systems in Nepal are running at low levels of both technical and allocative efficiency (Bhatta, Ishida, Taniguchi, & Sharma, 2006). Farm-level efficiency of use is also low due to use of the flooding method for irrigation, with high loss of water.
- Inappropriate crop choices also pose a problem for water security, for example when preference is given to water-intensive crops such as rice

and sugarcane even inwater-scarce areas.

- Financial constraints limit government investment in both new surface irrigation projects and the upkeep of existing projects.
- The incentive to invest in better water security for agriculture is limited by the fact that agricultural production in Nepal is unsubsidized and competes across an open border with subsidized production from India.

Hydro-energy security

Nepal has no fossil fuel reserves and limited potential for developing renewables like solar and windpower on a large scale, but it has large potential resources of hydropower. Ensuring water security for hydropower development mainly relates to ensuring investment for infra- structure to make the water available, as well as considerations related to competing uses, especially the environment. The main constraints are financial, political, and governance-related. As a small country, it is difficult for Nepal to generate enough capital to self-finance larger hydropower projects, even though it has successfully self-financed a number of small and medium-sized projects through the innovative mechanism of raising equity shares in the domestic markets. For larger projects, Nepal depends on bilateral or multilateral donors, and terms and conditions may not always be conducive for the hydropower producers, or the country. The market for surplus electricity is also a factor, with issues of cross-border trade and cooperation assuming importance, as well as questions of the larger political economy.

Urban and rural water security: the domestic sector

Nepal is the least urbanized country in South Asia but is now experiencing rapid urbaniza- tion, especially in Kathmandu and cities across the Terai. Urbanization creates new demands for water, and takes water away from agriculture and rural domestic needs to meet urban, residential, industrial and recreational needs (Narain, Khan, Sada, Singh, & Prakash, 2013).Urban water insecurity is a major emerging issue and is further complicated by issues of water pollution and solid waste management. The main impediment to urban water security is haphazard urban growth, which leads to the degradation of natural areas that used to recharge the groundwater. The other constraints are financial, managerial andtechnical.

Most of the rural population in the middle hills depends on springs for water for drinking and livelihood-related activities. In recent years, many of these springs haveslowed or dried up.

Governance in the new federal context

In 2015, Nepal promulgated a federal constitution and in 2017 moved to federalism in the form of one country with seven provinces and 753 local government entities. Nepal is in the process of restructuring institutional arrangements and harmonizing policy in this context, and many plans and policies at provincial and local levels have yet to be finalized. How the administration develops will determine how Nepal uses its rich endowment ofnatural resources. The aim is to be as inclusive as possible, while foreseeing, averting and managing potential conflicts. Natural resources and water management fall under the purview of all three tiers (federal, state and local governments; Table A2). Conflicts might arise in using natural resources and sharing the costs and benefits. There may be conflicting interests in the various uses of water resources in different provinces when supply and demand is uneven. Although the constitution indicates the jurisdictions of each level of government, it has yet to clarify their precise responsibilities. The Local Government Operational Act of 2017 made these categories clear for local government, but the distribution among provincial and federal governments is still being worked out. This should be further elucidated with the promulgation of the Electricity Act, Water Resources Act, Irrigation Act, and River Basin Master Plan.

Further drivers of water insecurity

A number of physical and socio-economic drivers of change could reduce water security in future. They can be divided into two main groups: physical (climate change and disaster risk); and socio-economic (population growth and competing water uses).

Climate change

The average annual maximum temperature for all Nepal increased by 2.4 °C (0.056 °C/y)over the 44 years from 1971 to 2014 (DHM, 2017). A reduction in glacier area of around 25% was recorded between 1980 and 2010 (Bajracharya, Maharjan, Shrestha, Bajracharya, & Baidya, 2014). Snowfall and glacier area affect the timing of river flow. Precipitation has also become more variable, with an increase in extreme precipitation events (Karki, Schickhoff, Scholten, & Böhner, 2017). These climatic changes can affect the quantity, quality and timing of water availability (MoPE, 2017) and thus reduce water security.

Climate change is likely to have a greater impact on water security in future. According to MoFE (2019), the climate change scenarios for Nepal suggest a significantly wetter andwarmer climate towards the end of the century. Both the average annual mean tempera- ture and the average annual precipitation are projected to increase (Figure A3). But pre- monsoon precipitation is projected to decrease, and extreme precipitation events to increase. These changes are expected to affect many sectors, including water, disaster management, energy, biodiversity, agriculture, health, urban planning, and livelihood- related activities (MoFE, 2019).

Disaster risk

According to MoHA (2018), more than 80% of the population of Nepal is at risk from natural hazards such as floods, landslides, windstorms, hailstorms, fire, earthquakes, and glacial lake outburst floods (GLOFs). Many of these natural hazards can affect water- related infrastructure and water availability. Hydropower especially can be affected by landslides, floods and GLOFs.

GLOFs in high-altitude areas have affected small and medium-size hydropower projects (International Centre for Integrated Mountain Development [ICIMOD], 2011). In 1981, the Dig Tsho GLOF destroyed the nearly com- pleted Namche Small Hydroelectric Project and caused further damage to downstream areas (ICIMOD, 2011). The small drinking water supply facilities in the mountains are affected by landslides and erosion, while in the downstream areas, irrigation and storage infrastructure is at high risk from sedimentation, floods and flash floods (MoPE, 2017).

Population growth

Nepal's population grew from under 10 million in 1961 to more than 25 million in 2011, and although this rate is slowing (the national average of 2.25%/y in 2001 dropped to 1.35%/y in 2011), the numbers are projected to grow for many years to come (Figure 3), with growth rates significantly higher in urban than in rural populations (Regmi, 2014). The larger population will probably need more water for drinking and domestic use, and

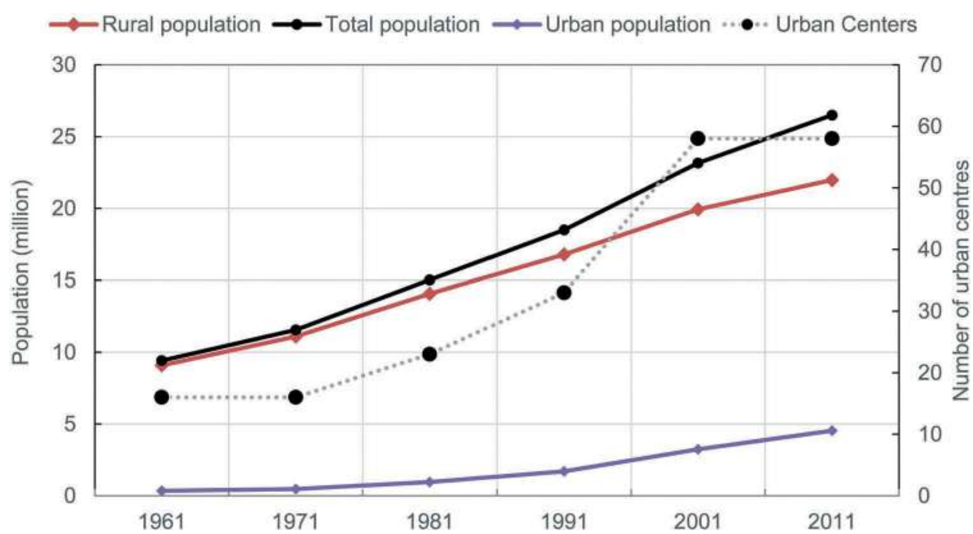


Figure 3. Growth in rural and urban populations in Nepal. Source: Data taken from CBS (2014).

for agricultural production to meet growing food requirements, and may bring higher vulnerability to water-induced hazards (Biggs et al., 2013). Economic growth, urbaniza- tion, globalization and higher standards of living are also increasing the per capita demand for water. Urban water security for drinking and other uses is likely to becomea major problem. Population growth in India, where part of the commercial food in Nepal is produced, is also likely to create pressure on food security in Nepal and increase domestic demand for water for food production (Biggs et al., 2013).

Competing water uses

Competition among water users for limited resources can also lead to water insecurity for particular users and uses. Although the Water Resources Act (1992) prioritized water usages to avoid conflicts between multiple usages, conflicts still emerge. The most common are between riparian hydropower and other user groups when the minimum flow requirement for other uses and the riparian ecosystem are not being met. The hydropower projects are meant to release the minimum environmental flow mandated by the environmental impact assessment, but monitoring is not strict, and the conditions are often not met (Shrestha et al., 2016). One reason is that releasing the mandated flow can result in economic loss for the project (Rijal & Alfredsen, 2015).

In the Middle Mountains, springs remain a major source of water for drinking and domestic purposes. Perhaps 13 million people rely on springs as a primary source (Central Bureau of Statistics, 2012). However, in recent decades competition for this water due tothe increasing population and changes in land-use patterns, together with infrastructure development and climate change, has led to a decline in spring flow: permanent springs have become seasonal, and some have dried up completely. The situation has become so grave that people are considering migrating away (Sharma et al., 2016; Tambe et al. 2012), and 28% of the population still has no access to safe supplies of potable water(Bartlett, Bharati, Pant, Hosterman, & McCornick, 2010).

Competing water uses can also lead to pollution of both surface and groundwater, with water used for cleaning and disposal of waste entering the supply for consumption and environmental use. Groundwater is polluted in many cities (Khatiwada, Takizawa, Tran, & Inoue, 2002) with more than 50% of the Kathmandu Valley affected by ground- water pollution (Shrestha, Semkuyu, & Pandey, 2016). Sewage disposal into freshwater also persists as a major issue in urban areas. The upsurge in city populations, coupled with poor urban planning, has led to large sewerage pipes being added to rivers as tributaries (Shrestha, Lamsal, Regmi, & Mishra, 2015). Other sources of contamination include factory waste and agricultural residues.

The water-energy-food (-environment) nexus

The WEF nexus can be defined as 'the very close links between these three sectors and the ways in which changes in one sector have an impact on one or both of the other sectors'(Nagle & Cooke, 2017), as well as how the sectors depend on each other. For example, water is an essential component in food and energy production; energy is used in numerous processes for supplying, treating and using water; and both water and energy are used for food production and transportation and along the entire agri-food supplychain. The multiple and competing uses for water, energy and food production mean there are important trade-offs that should be considered, often between sectors that are not coordinated. The nexus offers a conceptual approach to better understand and systematically analyze the complex interactions between the natural environment and human activities, and to work towards a more coordinated management and use of natural resources across sectors and scales (FAO, 2014). It can help identify and manage trade-offs and build synergies through our responses and therefore plays a vital role in livelihood security and socio-economic development (Rasul, 2014; Scott et al., 2019).

The key WEF nexus linkages in mountain regions are associated with hydropower generation and irrigation, both with important urban and rural implications, while in rural areas, the use of fuelwood for heating and cooking and rainfed agriculture for food production also play an important role (Scott et al., 2019).

Using the WEF nexus to address challenges to water security

Using the nexus concept in water security means unravelling the whole to reveal specific linkages between the sectors that affect water security, identifying challenges that have their source in sectors beyond simple water issues, and using these as a basis for developing specific recommendations for action. The nexus can be used to explore the interlinkages between the uses of water in different sectors (Rakhmatullaev, Abdullaev, & Kazbekov, 2017) and problems with ensuring water availability for different uses, such as hydropower and food production (Dhaubanjar, Davidsen, & Bauer-Gottwein, 2017; Rasul, 2014). Looking at water security from a nexus perspective can help us understand the wider implications and broaden the scope of interventions to include such things as water demand management, investment frameworks for public funding for improved surface irrigation, groundwater management, irrigation technologies and agricultural practices, as well as food procurement and trade policies (FAO, 2014).

In the following, we attempt to unpack the WEF nexus in Nepal by looking at each of the main water use sectors in turn – agriculture, energy, domestic and environment – and identifying the major challenges to water security and important linkages in the nexus with a view to identifying actions that can be taken to improve water security.

Agricultural sector

The agricultural sector is central to the WEF nexus. It is the main component in food production and a major user of water and energy. An estimated 65% of total agricultural production in Nepal comes from irrigated land (WECS, 2011). Agriculture accounts for 96% of total water withdrawal, with only 3.8% going to the domestic sector and a minimal amount to the industrial sector (AQUASTAT, 2018; WECS, 2011). However, despite considerable efforts by governmental and non-governmental agencies, a significant area of agricultural land remains under rainfed regimes (IMP, 2019; Poudel & Sharma, 2012), andmuch irrigated land has only seasonal access to water.

The main source of irrigation water is surface water from rivers, glacier and snow melt, lakes, wetlands and springs, although groundwater irrigation is increasing. Surface irriga- tion covers about 80% of the total irrigated area; the other 20% is irrigated through groundwater or mixed systems (FAO, 2016).

The irrigation command area varies from 90% of the system in the wet season to 25% in the dry season, with year-round irrigation less than 38%, due to the high seasonal variation in water availability (WECS, 2011). Extraction of water lies well below the potential. Less than 7% (15 BCM) of the total average annual river runof f of 225 BCM is used for irrigation (Bharati, Gurung, Jayakody, Smakhtin, &Bhattarai, 2014; WECS, 2005), and in the Terai alone, less than one-third of annual ground- water recharge is extracted for agriculture (Table 1). The ability to extract water for irrigation is directly linked to the availability of reliable and affordable electricity for pumping, and thus to the energy sector.

Water security in agriculture essentially means access to sufficient water for irrigation. The overall cereal yield in Nepal is 2.6 MT/ha, far lower than the regional and global average (World Bank, 2018), indicating overall low productivity. Access to year-round irrigation is vital to improve crop yields; at present most irrigation schemes function in the monsoon but not in winter and spring, when the need is greatest. A fully controlled irrigation system can have two or three times the yield of rainfed and monsoon-irrigated regimes (ADB, 2014).

Energy and industrial sectors

Energy sector

Water is the primary input for hydro-based energy generation, which provides 99.5% of Nepal's electricity production (Nepal Electricy Authority, 2015). At the same time, hydro- power is itself important for water security, as energy is used to pump water to meet various water demands, including domestic and industrial demand in the cities, and agricultural water demand in rural areas. Nepal has a great potential for hydropower development, thanks to the high precipitation and steep topography, along with per- ennial rivers fed by glaciers and snow in the high mountains. It is estimated at 83,000 MW (Shrestha, 2016), of which 43,500 MW is thought to be economically and technically feasible (Dixit, 2008). Only a fraction of this has been exploited so far. The present generation is close to 1050 MW, with a further 7950 MW planned in 209 projects licensed for construction and now at different stages of completion (Department of Electricity Development, 2019). Nepal still imports 450 MW of power from India and produces an additional 54 MW from diesel fuel (MoEWRI, 2018).

Despite its high per capita potential for electricity production, Nepal has a supply deficit, and its per capita consumption is among the lowest in Asia. Thirty percent of rural households still lack access to electricity (MoF, 2018). Most of the population still relies on traditional energy sources for cooking and heating, and these sources com- prise 69% of total energy consumption. Electricity demand is increasing by approxi- mately 7% per year (Nepal Electricy Authority, 2015), mainly for rural electrification, industry and domestic use. The demand forecast for 2033 is around 5785 MW (Figure A4).

Hydropower is seen as a potential medium to support the national economy, but could also provide substantial revenue at the local level. Similarly, access to energy can open up opportunities for many other electricity-dependent sectors. But the country faces several challenges in hydropower development, a major one being seasonality. Production is largely dominated by run-of-the-river projects, which produce less during the dry season, when demand is highest, and a surplus in the rainy season, when demand is lower (Sharma & Awal, 2013). Only one of the 73 operating projects is a storage type (Department of Electricity Development, 2019). It is estimated that even if all of Nepal's potential is exploited, dry-season production would amount to only 11,300 MW (Shrestha, 2016). The seasonality is likely to be aggravated by climatic and other environmental change.

Achieving water security in the hydropower sector implies making sufficient water available in the form of run-of-the-river flow or storage infrastructure to produce the full electricity requirement.

Industrial sector

Industrial growth outside the energy industry is very limited. Industry contributes only 5.4% of GDP, down from 10% 20 years ago (MoF, 2018), and represents only 36% of total power consumption, less than household consumption (45%). Investment is low, with only 54% of investment in the industrial sector going to energy-based industries (MoF, 2018). To some extent, lack of industry to provide a consumer base for power means lack of incentive to increase electricity production, while lack of reliable electricity suppy means lack of incentive for industrial growth. However, as energy projects come on line, there is likely to be an upsurge in industrial growth, leading to economic development, which will further stimulate an increase in hydroelectricity production. But this will also increase competition among water users, as both hydropower production and industries themselves will have higher water requirements, in compe- tition with the domestic water use, agriculture and the environment. It may also affect water supply through increased pollution. Achieving water security for industry will require increasing the total volume of available water and/or increasing water use efficiency.

Domestic sector

The domestic sector predominantly uses water for drinking and sanitation and to supply livestock. As of the last census, 85% of Nepal's population had access to basic water supply services, and 62% to basic sanitation facilities (Central Bureau of Statistics, 2012). Domestic water is mainly supplied through surface or groundwater schemes implemen- ted by government and community-based organizations. But the water supply remains strongly seasonal, with more limited access to drinking water in the dry season.

In the mid-hills, springs are the main source of domestic water. In many cases, spring water is collected in storage tanks and accessed via taps. In some cases, stored water is pumped to a higher level and supplied by gravity flow to individual houses or community taps. Spring discharge is also used for small-scale irrigation on hill slopes. But there is increasing evidence that springs are drying up; they are becoming seasonal, and their discharge is dropping. A study by Sharma et al. (2016) found that one-third of the springs in Kavre District have dried up, and Chapagain, Ghimire, and Shrestha (2019) found that spring discharge had declined by over 30% in the last 30 years in one mid-hill region.

In the plains of the Terai, the major source of drinking water is groundwater, which meets close to 90% of the drinking water demand in the region (ADB and ICIMOD, 2006). Only 14% of households have access to piped water, while about 80% pump water from the 800,000 shallow and drinking tubewells in the region (Kansakar, 2005; MoWSS, 2016; Sarwar & Mason, 2017). Tubewells and shallow wells are commonly used to supplement the limited supply of municipal water in cities, including Kathmandu, where groundwater levels have dropped considerably due to excessive use to meet the increasing demand of the growing population (Pandey, Chapagain, & Kazama, 2010).

Domestic water supply has close connections with energy, agriculture and the envir- onment, as provision of domestic water is strongly affected by the energy available for pumping, by land use and land cover change affecting recharge, by extraction of surface and groundwater for competing uses, and by availability of storage infrastructure. Ensuring domestic water security will require addressing all of these.

Environmental sector

The entire ecosystem depends on water for its function, but is itself also a major compo- nent of water supply, for example through capturing runoff, providing storage in soil and root systems, and recycling water to the atmosphere. A healthy ecosystem is essential for the water supply, and water is essential for a healthy ecosystem. Nepal's 2005 National Water Plan made supporting water-dependent ecosystems a major priority (WECS, 2005). These ecosystems can be disturbed by upstream water uses, infrastructure development and pollution. For example, if water is diverted from a stretch of river for hydropower production, the lack of year-round flow along that stretch can affect the local ecosystem, especially fish and other riparian life. Environmental flow assessments provide an under-standing of the amount of flow required in a river system to support the freshwater ecosystem. They should include an understanding of wet- and dry-season flows, natural high flows, extreme low flows, floods and interannual variability, all of which determine the flow regimes of river systems (Manandhar, 2016). In urban areas, rapid unplanned urbanization with direct disposal of household waste and sewerage into rivers has also harmed river ecosystems (Karn & Harada, 2001). Discharge from industry can also affect water quality and dependent ecosystems in downstream areas.

Maintaining water security for the environment means ensuring that sufficient flows of water are retained to maintain ecosystem function. Plans for water extraction for use in other sectors must be balanced against the need to protect ecosystems and their dependent biodiversity.

Improving water security in Nepal

Water security is still a key issue for many people in Nepal, in spite of the theoretically high physical availability of water. The impediments to water security result mainly from economic and institutional factors rather than physical water scarcity, as outlined in the previous sections. The issues and potential measures to address them can be summar- ized for each water use sector, but specific measures will also affect other issues and lead to outcomes in other sectors. Furthermore, the move towards integrated manage- ment of water resources and approaches to water governance will also play an impor-tant role in achieving water security. These aspects are discussed in the following sections.

Addressing sector-specific issues

The issues identified in specific sectors from a range of studies over many years are summarized in Table 2, together with potential measures to address them. The major issues are presented in more detail in the preceding sections, and others are included in the table for completeness. It is beyond the scope of this article to go into details on all these topics; rather, we provide an overview, with details especially of issues that benefit from using a nexus approach. Although presented sector-wise, the issues and measures take into account the linkages in the WEF nexus following the water security framework presented in Figure 1.

Unpacking nexus linkages to improve water security

The nexus among groundwater, energy, and irrigation has been highlighted in a number of studies (Mukherji, 2007; Shah, Scott, Kishore, & Sharma, 2004). One of the clearest results of the analysis using a nexus approach to water security is the recognition that in Nepal this nexus offers one of the most accessible opportunities to address water security, which is to increase the water available to agriculture by increasing the extraction and use of groundwater to levels up to annual recharge through increased production of afford- able energy. The approach is made possible by the high rainfall and recharge and the aquifer

Table 2. Measures to improve water security in different sectors.

Table 2. Measures to improve water security in different sectors.

Sectors	Major issues	Water security measures
Agriculture mountains	• Access to water in and hills • Lack of water for irrigation in the plains • Energy availability and access for water extraction • Water loss • Climate variability and change • Drying spring sources • Access to markets	• Increasing water supply through inf and rejuvena- tion of springs • Groundwater pumping using grid electr off-grid solar power • Small and local-scale water storage harvesting • Inter-basin water transfer • Irrigation technologies, farm technologies (demand management) • Formal and informal water markets
Domestic water	• Access to drinking • Drying spring sources • Urban water supply systems • Sanitation	• Spring conservation and management hydro- geological and community ap small and local-scale water stora rainwater harvesting • Increasing water supply (inter-basin water • Sustainable use of groundwater in an urba • Multipurpose water storage infrastructure • Public–private partnerships
Hydropower availability	• Seasonality of water • Hazard risks (landslides, glacial lake outburst floods) • Sedimentation • Climate change and variability (e.g. extremes) • Maintaining environmental sustainability	• Considering downstream water u irrigation, environ- ment and domes uses • Multi-hazard risk assessment • Sustainable use of groundwater • Reducing pollution and maintaining the e • Optimization of industrial processes to r use • Regulatory framework in place with prop
Industry	• Access to water • Overuse of groundwater • Water pollution and contamination	• Environment management plan based environmental impact assessment • Promoting aquatic movement a infrastructure • Effective monitoring and compliance

conditions, and the resulting increase in both production and the productivity of agriculture would also improve food security. Access to irrigation can be boosted primar- ily through investments in the energy sector to exploit the country's potential for hydro- power and increase the coverage of the energy supply network, and on a smaller scale through investment in solar power irrigation systems. The rationale behind this approach and the potential benefits are presented in more detail below, followed by a range ofother actions to improve water security.

Exploiting the nexus between groundwater and agriculture in the Terai The net area of cultivated, irrigated and irrigable land in the Terai, hill and mountain areas ofNepal is shown in Table 3. At present only 1.1 million of the 3.5 million hectares of cultivated land (31%) is irrigated, but a further 1.3 million hectares could be irrigated if water can bemade available (IMP, 2019). Nearly half of all cultivated land and most irrigable land lies in the fertile alluvial plains of the Terai, popularly known as the granary of Nepal. Close to 60% of potentially irrigable land already receives some irrigation, about half from surface water, a quarter from groundwater, and the rest from conjunctive use of surface and groundwater, but more than 600,000 ha of land that could be irrigated still supports only rainfed agriculture. Furthermore, half of the present irrigation in the Terai is through surface irrigation schemes which are mostly fed by small and medium-sized rivers. But there is considerable fluctuation in the net command areas between the monsoon and dry seasons, with large areas receiving no water in the dry season and thus unable to grow winter and

Table 3. Irrigated and potentially irrigable land in Nepal (source: IMP, 2019).

	Terai		Hills		Mountains		
Category	ha ('000)	%	ha ('000)	%	ha ('000)	%	Total ('000)
Cultivated agricultural land	1,594	44.8	1,566	44.0	401	11.3	3,561
Potentially irrigable land	1,480	65.3	627	27.7	159	7	2,265
Present area irrigated							
Surface water	*434*		*170*		*41*		*654*
Conjunctive use	*207*		–		-		*207*
Groundwater	*226*		*8*		-		*234*
Total	866	79.8	178	16.4	41	3.8	1,085
Net potential irrigable land	613	51.4	448	38.5	118	10.1	1,180

spring crops. Even in areas growing monsoon crops, timely onset of the monsoon is critical, as there may not be enough irrigation water if the monsoon is late.

The estimated annual recharge of groundwater in the Terai is 8.8 BCM, but less than a quarter of this is currently extracted, leaving a potential balance of 6.9 BCM that could be used for irrigation (Table 1). More water could be used both to increase the overall irrigated area and to increase the seasonal coverage of irrigation in areas already receiving some water through conjunctive use of surface and groundwater in the dry season. Year- round irrigation would help boost both yields (which are among the lowest in the plains areas of

South Asia) and cropping intensity. A recent study suggests that with sufficient water, the plains of the lower Gangetic basin can support 2.5 crops per year without threat of long-term depletion, compared to the 1.2 to 1.5 crops per year grown in the Terai at present (Shah, Rai, Verma, & Durga, 2018). Not only could more crops be planted, water availability would facilitate effective use of other agricultural inputs such as fertilizers, pesticides and improved crop varieties, also leading to higher crop productivity and production. Use of such inputs in conditions of water stress may not improve yields and can even be counterproductive (Khunthasuvon et al., 1998).

There are two main factors to be considered when proposing to increase groundwater extraction to support agriculture: the sustainability of extraction amounts, and how to ensure a regular and uninterrupted supply of energy for extraction.

Sustainability of extraction was explored using calculations based on expansion of irrigated area by the full potential of 613,000 ha in Terai (Table 4). The crop water requirement differs from crop to crop, and by variety, soil type, and season. However, a conservative estimate shows that on average 5000–10,000 m^3/ha of water is required to cultivate cereal crops in South Asia (Facon, 2000; Haavisto, Santos, & Perrels, 2018), giving a maximum of 613 MCM for summer (dry-season) rice. Although water is abundant for growing rice in the monsoon season, delayed onset of the monsoon can require supplementation with groundwater, estimated at 307 MCM. Finally, irrigation would enable cultivation of winter crops such as wheat and barley, requiring a further 490 MCM. The total 1.4 BCM required to cultivate three crops per year on all the potentially irrigable land

Table 4. Estimated groundwater requirement to irrigate irrigable land in the Terai (author's calculation based on Facon, 2000; Haavisto et al., 2018; IMP, 2019).

Crop	Water requirement (MCM) for 613,000 ha of irrigated land
Summer rice	613
Supplement for monsoon rice	307
Winter crops (e.g. wheat, barley)	490
Total	1,410

in the Terai is still only 20% of the calculated recharge surplus and thus within sustainable amounts. This would leave a considerable potential for additional irrigation of land which already receives some surface irrigation but not year round, enabling additional crops to be grown in winter and summer (before the monsoon).

Exploiting these groundwater resources requires access to a reliable and affordable supply of energy. This can be achieved primarily by exploiting a greater part of the country's potential resources for hydropower, as outlined under 'Energy Sector' earlier. A rough estimate is that the energy requirement would be on the order of 153,000 kWh per day, equivalent to 15 MW (to pump 1410 MCM per year from a depth of 10 m at 70% efficiency). On a smaller scale, solar-powered irrigation systems can be used to help smallholder farmers irrigate their lands and reduce dependence on the national grid. Solar pumps have an advantage over hydroelectricity in many rural areas where electricity infrastructure is still limited, especially to farmland. Linking farms to the electricity grid increases costs, and the fragmented parcels add complexity. Shah et al. (2018) suggested that solar-powered pumps are a good option for groundwater pumping in the Terai, while in future a combination of grid electricity and solar power could help maximize oppor- tunities (Mekhilef, Faramarzi, Saidur, & Salam, 2013). Constructing large irrigation systems takes a huge amount of resources and a long time, but a distributed energy approach with local pumping using hydropower and solar power irrigation systems could provide immediate benefits.

The potential additional revenue from irrigation of the additional 613,000 ha was estimated from the difference between the market price of the average rainfed yield of monsoon rice cultivated at present, and the value of three crops (monsoon rice, summer rice and winter wheat) grown over the whole area with year-round irrigation (Table 5). An estimated additional USD 2.2 billion (NPR 220 billion) could be generated as direct gross benefit. A conservative estimate puts production costs at 50%, based on values of 40–55% reported for rice-wheat-dominated agriculture in the Indo-Gangetic Plain (Jat et al., 2014). Thus, net direct benefit could be on the order of USD 1.1 billion. This is equivalent to 4.5%of national GDP in 2017. There would also be indirect benefits, such as local employment, and gains in the markets for inputs and outputs. Further, Nepal imports a huge amount ofcereals, vegetables and fruit, mostly (over 70%) from India. Currently, Nepal exports agricultural commodities equivalent to USD 190 million and imports agricultural com- modities equivalent to USD 950 million, a net deficit of USD 860 million (calculated from World Integrated Trade Solution, 2018). If water security in agriculture is achieved, the additional agricultural production could easily fill the trade deficit gap and help achieve food security. Irrigated summer crops generally have higher productivity than monsoon crops because of the greater solar radiation, and are also less vulnerable to diseases and pests. Where markets are accessible, as in the Terai, farmers also have the opportunity to grow higher-value cash crops like fruits and vegetables. For example, solar pumps introduced for irrigation in Saptari District of Nepal increased the cropping intensity and yields and helped farmers switch from cereal to vegetable production and fish farming, with a potential increase in farm income (Mukherji et al., 2017; personal com- munication, Nabina Laamichhane, ICIMOD, 18 April 2019).

Table 5. Potential yield gains following groundwater irrigation of the 613,000 hectares in the Terai growing rainfed crops at present.

Crop	Average rainfed yield (MT/ha)	Maximum irrigated yield* (MT/ha)	Additional yield with irrigation (MT/ha)	Additional production with irriga- tion (1000 MT)	Market price** (NPR / kg)	Current rev- Enue (NPR millions)	Revenue with irri- gation (NPR millions)	Additional revenue (NPR millions)
Summer rice (April to June)	0	3.9		2391	47	0	112,000	112,000
Monsoon rice (June to October	3.3	3.9	0.6	368	63	127	150,000	23,000

)								
Winter wheat (November to March)	0	2.9	1,778	48	0		85,000	85,000
Total			4,535			127	347,000	220,000 (USD 2.2 billion)

Source: *FAOSTAT (2019); **MoALD (2017); MoAD (2019).
Notes: Price of coarse rice is for summer rice, and fine rice for monsoon rice; USD 1 = NPR 100 in 2017.

Other benefits and approaches

The potential for using power to pump groundwater on a large scale increases the overall incentive for increasing hydropower production. This is likely to generate an excess of power, offering further opportunities, such as using electricity in agricultural storage and processing, developing power-dependent industries, increasing domestic consumption through innovative uses such as cooking in place of LPG and wood, and selling to nearby markets, all of which can contribute to revenue generation, employment and local development.

Using groundwater can also have other direct and indirect benefits. One is the potential to reduce the intensity of flooding in the monsoon season, as suggested by the Ganges Water Machine hypothesis of Revelle and Lakshminarayana (1975). The hypothesis is that lowering the water table in the pre-monsoon season could allow higher recharge during the monsoon, using water that would otherwise contribute to flooding. It was suggested as a mechanism for storing excess water in the monsoon season in aquifers in the eastern Gangetic basin where the terrain is too flat to construct large water storage structures. The hypothesis has been tested using empirical data for West Bengal (Mukherji, Banerjee, & Biswas, 2018), Bangladesh (Shamsudduha, Taylor, Ahmed, & Zahid, 2011), and India as a whole (Chinnasamy, 2016). All these studies found that lowering the shallow aquifers before the monsoon allowed more water to infiltrate at the beginning of the monsoon season. Thus, withdrawing large amounts of water from the aquifers in the Terai during the dry winter and summer seasons could have the additional benefit of reducing flooding intensity when the excess water is used for recharge. This implies that care must be taken to ensure that the recharge pathways are maintained and possibly proactively managed, which has further implications for farming practices and infrastructure construction.

There are other methods that can be used to increase water supply. For example, the Nepalese government has made a dedicated effort to increase water supply through inter-basin transfer by bringing water from the Melamchi watershed to the Kathmandu Valley (Bhattarai, Pant, & Molden, 2005). At a local scale, promoting rainwater harvesting and artificial recharge of shallow and deep aquifers could also ease the water supply, at least during the monsoon and the months right after (Shrestha, 2009). Possibilities for improving water security include a dedicated spring conservation programme, water storage of various sizes to hold rainfall and recharge groundwater, and springshed management approaches (Shrestha et al., 2017).

In urban areas, the nexus approach makes it clear that integrated solutions are needed that look at issues like drinking water provision, stormwater management, and manage- ment of wastewater and solid waste. Improving urban water security requires that urban development planning consider supply and demand management, maintaining natural and human-made recharge structures and strict control of haphazard groundwater exploitation.

Promoting an integrated approach to water management through governance: IWRM and the nexus approach

Water insecurity in Nepal, in the face of abundant theoretical availability, is linked to problems of supply and demand, in which water governance plays an important role (Neupane, 2011). Using water resources sustainably requires water governance that uses a holistic and integrated approach to the management of water and related resources. Nepal's Water Resources Strategy (WRS) of 2002 (WECS, 2002) and National Water Plan (NWP) of 2005 (WECS, 2005) envisage using IWRM. WRS 2002 was developed using a participatory approach with social, economic and environmental sustainability principles, with the overall aim of improving the living standard of the Nepalese people in a sustainable way. The NWP 2005 was developed to implement the activities identified in WRS 2002. The NWP recognizes the broad objectives of the WRS and lays down short-, medium- and long- term plans for the water resources sector, including investment and human resource development. It attempts to address environmental concerns, and includes an environ- mental management plan. This plan is designed to maximize positive impacts and minimize or mitigate adverse impacts in line with environmental sustainability concerns. The NWP also adopts IWRM principles for the development of water resources in a holistic and systematic manner. However, although the WRS and the NWP were formulated one-and-a-half decades ago, implementation of IWRM has been slow. As suggested by Suhardiman, Clement, and Bharati (2015), some of the prominent challenges to implementation are lack of coordination among government institutions, lack of clarity on the mandates of key institutions, and lack of strong political will, with water issues fragmented among different ministries in the past. This also indicates that water governance is a key issue.

Although IWRM promotes an integrated approach to water, land, and other related resources, in practice it has only been implemented at large scales and has only been tested in a few selected projects; the current challenge is to demonstrate improvement of existing water management practices (Biswas, 2008). Suhardiman et al. (2015) suggest that the current discourse on IWRM in Nepal highlights the need to shift the emphasis from national policy formulation to local, adaptive, pragmatic approaches to IWRM. It remains to be seen how IWRM can help solve the water crisis, support water-based adaptation and livelihoods, and deal with climate-change-related challenges at different scales.

The WEF nexus and IWRM are linked, but they have different approaches and focus (Ringler, Bhaduri, & Lawford, 2013). IWRM is specifically water-sector-oriented, and is less concerned with linkages with other sectors, whereas the nexus approach not only takes into account all the different users and uses of water, it specifically considers the linkages among water, energy and food and looks at the trade-offs and synergies, and the possibilities they offer for increasing water security through actions in other sectors. Although IWRM attempts to involve all sectors of water management, the nexus approach treats water, energy and food equally and also considers their interdependencies (Rasul et al. 2014). Essentially, IWRM deals with the entire life cycle of water, and provides tools for implementation, while the nexus approach deals with the life cycle of water and other related processes, including energy, land and food, and suggests pathways for intervention.

Biggs et al. (2013) suggested that while the principles of IWRM and water security have some complementarity, given the limited uptake of IWRM to date, perhaps an alternative framing such as the nexus-based approach is required for effective water management and governance. The scope for greater merging of nexus thinking within IWRM has been discussed in some detail by Benson, Gain, and Rouillard (2015). They conclude that:

> Scope exists within current IWRM practice to better shift water governance from prioritizing *inter alia* demand management and resource efficiency towards securing an acceptable quality and quantity of water for all users and ecological protection: all key objectives of IWRM. A critical issue is how best to balance water, agricultural, and energy security at multiple scales. Current nexus arguments still remain highly ambiguous on this subject, inferring a need for strategic policy guidance and institutional structures across multilevel governance. (p. 768)

This study illustrates how the nexus approach can be used to identify pathways to water security that originate in actions and policies related to other sectors, specifically the encouragement of generation of hydropower to provide affordable energy for pumping groundwater to support agriculture. Exploiting the groundwater requires cross-sectoral policy coherence among the agriculture, energy and water sectors. The IWRM and IRBM approaches which the Nepal government has adopted as policy need to be broadened to recognize the nexus between water, energy and food production and security. Further, rainfall and temperature patterns are likely to be affected by climate change, and additional adaptive water management practices may be necessary to help achieve water security. The nexus approach has also been discussed as an important tool in adaptation to climate change (Rasul & Sharma, 2016). The framework proposed in Figure 1 could be helpful for integrating these different aspects. The ultimate aim of the WEF nexus approach is also to use the framework as a tool for assessing environmental livelihoods (Biggs et al., 2015).

The key linkages identified using the nexus approach will vary according to the regional, national and local conditions in the area of interest. Particularly from a water perspective, a river basin approach is crucial, as it is fundamental in terms of the linkages among water resources and water uses (Lawford et al., 2013) in both upstream and downstream areas (Nepal, Pandey, Shrestha, & Mukherji, 2018; Nepal et al., 2019). As with IWRM, the aim should be to broaden the approach of IRBM to encompass WEF linkages at the river basin level (Hooper, 2003). The governance framework needs to be designed in such a way that the intersectoral linkages are not only taken into account but fundamental to policy development and the identification of interventions. The recent creation of a Ministry of Energy, Water Resources, and Irrigation at the central level is a promising step in this direction and should be helpful for settling water, energy and irrigation-related issues at the provincial and local levels.

Conclusion

Water security continues to be a major concern in Nepal in all sectors, despite the abundance of annual rainfall. Water demand is likely to increase with population growth, urbanization, industrialization, more intensive agriculture and increasing standards of living, while water supply may itself be affected by changing climatic patterns.

Addressing water security requires new and innovative approaches based on our better understanding of water not as a ‘stand-alone’ good but as one aspect of an integrated system. The WEF nexus concept recognizes that water, energy, food and other land-based resources are interlinked and have complex interactions, leading to synergies and trade-offs. The present study shows how using this concept to investigate linkages to water security in different water use sectors can help identify specific approaches that can be used to address water security in other sectors. Specifically, Nepal’s plains contain large and mostly untapped reserves of renewable groundwater, which can be used to increase the water supply to irrigated agriculture, and this can be achieved primarily by introducing policies to promote development of Nepal's hydropower potential to provide reliable, affordable energy for pumping. The estimated potential direct economic gain from providing year- round irrigation to the unirrigated agricultural area in the Terai is on the order of USD billion, equivalent to 4.5% of Nepal's annual GDP in 2017. There would also be indirect benefits in terms of generation of local employment, boost to markets, and reduction of imports. In terms of the nexus, the energy sector would itself benefit from the incentive to increase hydropower production provided by the enlarged consumer base for pumping, while disaster risk reduction could benefit from the reduced flooding intensity when the excess water is used to recharge the aquifer. Other linkages, such as promotion of solar power for pumping, with benefits to smallholders and industry, are mentioned and remain to be further explored.

The study also laid out the main problems in other water use sectors, including the drying up of springs in the mid-hills, which impacts domestic water use as well as livestock farming, and the problems resulting from rapid urbanization, whereby urban water security is threatened by the haphazard and uncontrolled use of groundwater, combined with reduction in recharge due to construction. Some measures to address these, such as springshed management, are suggested, but the detailed linkages within the nexus play a less marked role, and more detailed analysis lies beyond the scope of this article.

Nexus considerations also highlight areas for caution. For example, using groundwater to address water security issues in any sector requires also assessing the potential environmental impact and level of sustainable extraction, and ensuring that recharge pathways are protected and pollution is avoided. In particular, pumping of large quantities of water for industrial and urban use needs to be carefully monitored and controlled by government agencies. Environmental use needs to be given special consideration, in terms of both water-dependent ecosystems and biological diversity.

Water governance plays an important role in addressing water security. Using water resources sustainably requires water governance that uses a holistic and integrated approach to the management of water and related resources. Nepal's water policy envisages using IWRM, but implementation of the approach has been slow. The nexus and IWRM concepts are linked but have different approaches and focus. We suggest that a nexus-based approach is required for the implementation of effective water manage- ment and governance, and that IWRM and IRBM approaches need to be broadened to take into account the nexus between water, energy, and food production and security. The new federal structure poses a further challenge in this, with responsibilities and provisions at different levels still to be finalized. This includes a need for a clearer jurisdiction for all three tiers of the government. However, it also offers opportunities as new policies and implementation pathways are formulated at different levels of govern- ance. The creation of a combined Ministry of Energy, Water Resources, and Irrigation at the central level is a promising first step.

1.10 Resources Mobilization in irrigation Sector
1.11 External Financing in Irrigation Sector
बहुउद्देश्यीय सिंचाई आयोजनाको बिकासका विधी एवं प्रक्रियाहरु

नेपाल सरकारले हालसम्म बहुउद्देश्यीय सिंचाई आयोजनाहरुको विकास पुर्णरुपमा सरकारको आफ्नो लगानी तथा दातृ निकायको ऋण सहयोगमा गर्ने गरेको छ संघियता लागू भए तिनै | तहका सरकारहरु विच राजस्व बाँडफाँडगर्नुपर्ने हुन्छ यसले आगामी दिनमा राज्यले ठुला | तथा बहुउद्देश्यीय सिंचाई आयोजनाहरु विकासको लागि पुँजी जुटाउन बैकल्पिक उपायहरु | ति मध्ये निम्न मोडेलहरु हुन् सक्छ | अबलम्बन गर्नुपर्ने आवश्यकता देखिन्छ

- नेपाल सरकारको आफ्नो लगानी
- वैदेशिक ऋण
- निजी क्षेत्रको लगानी
- EPSF)EngineeringprocurementconstructionFinancing(
- सार्वजनिक निजी साझेदारी
- विकास समिति कम्पनि मोडेल आदि
  - नेपाल सरकारको आफ्नो लगानी

यस्तो लगानीमा दाताको कुनै शर्तहरु बाध्यकारी नहुने साथै आफ्नो हिसाबले विकास गर्न सकिन्छ र वित्तीय हिसाबले आकर्षक नभएका आयोजनाहरु तर आर्थिक सामाजिक दृष्टिकोणले अत्यन्त आवश्यक हुन सक्ने आयोजनाहरु विकास गर्न सकिने

तथापी, लगानीको सिमितताको कारणले धेरै आयोजनाहरुको विकास एकै पटक गर्न नसकिने सथिया बजेट विनियोजन न्यून हुने हुनाले आयोजना सम्पन्न गर्न लामो समय लाग्ने र लागत वृद्धि समेत हुनसक्ने

वैदेशिक ऋण

दातृ निकायहरुबाट सहुलियत ऋणव्यापारिक ऋण अनुदान लिएर प्राथमिकता प्राप्त जलस्रोतका , निर्माण गर्न सकिने तर तल्लो तटीय असार पर्ने जलस्रोतको ठुला बहुउद्देश्यीय आयोजनाहरु क्षेत्रमा दातृ निकायले लगानी नगर्ने, दातृ निकायको अनावश्यक शर्तहरु रहने, आयोजनात्यरिमा धेरै समय सहित निर्माणमा अझै बढी समय लाग्ने, समाजिक वातावरणीय , तथा आर्थिक विश्लेषण गर्न, तथा अनुपालन Complianceमा लामो समय र खर्च लाग्न सक्ने

निजी क्षेत्रको लगानीमा)VGFसहित(

EPCF

सार्वजनिक निजी साझेदारीमाBOT ,DBOetc.

बहुउद्देश्यीयजलाशययुक्त आयोजनाहरु प्रतिफलका हिसाबले आकर्षक हुने हुँदा , बैदेशिक लगानी बाट निर्माण गर्न सकिने तर त्यसका लागि /निजी क्षेत्रको प्रत्यक्ष लगानी

सरकारले जग्गा प्राप्ति, वातावरणीयबाढी नियत्रण ,, सिंचाई जस्ता बिषयबस्तुमा VGFसमेत आवश्यक नीतिगत र कानुनी प्राबधानहरुलाई समय सापेक्ष गर्नुपर्ने हुन् सक्छ।

कम्पनि मोडेल

कम्पनि ऐन अनुसार कम्पनि गठन गरी निजिक्षेत्रसिंचाई ,तिनै तहको सरकारको लगानी , विभाग, विधुत बिभाग को लगानी र स्वदेशीविदेशी बैंक बाट ऋण आदि व्यवस्था गरी / बहुउद्देश्यीय आयोजनाहरु निर्माणगर्ने सकिने

८. विकास समिति

बहुउद्देश्यीयसंचालन ,जलाशययुक्त र अन्तर जलाधार जलस्थानान्तरण आयोजनाहरुको विकास , र व्यवस्थापन विकास समिति गठन गरेर जसमा सरकारको पूर्ण लगाई तथा वैदेशिक दातृ निकायबाट ऋण लिई समेत गर्न सकिने

बहुउद्देश्यीय आयोजनाको संचालन तथा व्यवस्थापन विधि

- सिंचाई संरचनाहरुको संचालन र व्यवस्थापन

आयोजनाको बाँध, हेडबोर्कसमूल नहर ,, शाखा नहरहरू मध्ये सिंचाई नीति अनुरुप ठुला सिंचाई आयोजनाहरुमा मूलनहर भन्दा तलका संरचनाहरु जलउपभोक्ता संस्थाले संचालन, मर्मत तथा सम्भार कार्य गर्न सकिने

जल विधुत गृहको संचालन र व्यवस्थापन विकल्पहरु मा जलस्रोत तथा सिंचाई विभाग आफैले वा बिधुत प्राधिकरणलाई हस्तान्तरण गरेर

- लिज, संचालन तथा हस्तान्तरण
- व्यवस्थापन करार
- स्वामित्वकरण, संचालन र हस्तान्तरण
- विकास समिति
- कम्पनि मोडेल आदि

प्रचलित ऐन नियम नीति मा निजी क्षेत्रको संलग्नताको मौजुदा व्यवस्था

१. जलस्रोत ऐन/नियमावली

- अनुमतिपत्रको माध्यमबाट योजनानिर्माण गर्न सकिने.
- सेवाविस्तार गर्न सकिने,
- सेवाशुल्कलिन सकिने,
- शुल्कनतिर्नेलाई सेवाबाट बन्चित गर्न सकिने,
- आवश्यकजग्गा सरकारले प्रचलित ऐन बमोजिमउपलब्ध गराईदिने.
- संरचना सुरक्षाव्यवस्था,
- सरकारले विकास गरेका योजनाहरुलाई शर्तहरु तोकीउपभोक्ता संस्थालाई। हस्तान्तरण गर्न सकिने,
- स्वदेशी, विदेशी,कम्पनी संस्थावाव्यक्तिसंग करार गरी जलस्रोतको विकास,उपयोग र सेवाविस्तार गर्न गराउन सकिने,

- अनुमतिपत्रनविकरण गर्न सकिने,
- बिक्रिवा हस्तान्तरण गर्न सकिने प्रावधानहरु,
- विवाद निरुपणको लागिजाचबुझ समितिको व्यवस्था,

२. WRS/NWP

- Efficiencyबढाउन र CostRecovery को लागिनिगी क्षेत्रको सहभागिता बढाउने,
- ठुला सिंचाइ योजनाहरुको ManagementContract गर्ने,
- उर्जा,जलस्रोत तथा सिंचाइमन्त्रालयमानिगीकरण Cell को व्यवस्था,
- NWPकार्यान्वयनको लागि १५% बजेट निजीक्षेत्रबाट लगानी गराउने Target

३. सिंचाइनियमावली

- संयुक्तव्यस्थापनमा रहेको वा नेपाल सरकारद्वारा संचालित सिंचाइ (JMIS/AMIS) प्रणालीभित्रका नहर, शाखानहर,प्रशाखानहर तथाकुलेसोको मर्मत सम्भार तथा संचालन को जिम्मेवारी कुनै व्यक्ति, WUA/NGOलाई ३५ दिने सूचनाप्रकाशित गरी प्रतिस्पर्धाको आधारमाकबुलियत गरी आंशिकवा पूर्णरुपमादिन सकिने-(नियम43 क)
- स्थानीय निकायबाट मागभइआएमाप्राविधिकक्षमता स्रोतको व्यवस्था समेतलाई हेरी शर्त सहित (निर्देशिकामाउल्लेख भएबमोजिम) व्यवस्थापनको जिम्मेवारी हस्तान्तरण गर्न सकिने -(नियम43 ख)

४. सिंचाइनीति २०७०

- सिंचित क्षेत्रको घोषणा, (दफा1.5.2)
- सिंचाइ प्रणालीको निर्माण,संचालन र व्यवस्थापनमानिजी क्षेत्र, सहकारी तथा सामुदायिक संगठनलाई सहभागी गराउने, (दफा1.5.3 र 1.6.13)
- व्यापारिक उपयोगको लागिप्रचलितकानुनबमोजिमअनुमतिपत्रलिनु पर्ने,(दफा1.6..9)
- विदेशीतथा स्वदेशीनिजीलगानीकर्तालाई प्रचलितकानुनबमोजिमआकर्षित गरिने (Built, OperateandTransfer -BOT-2063) (दफा1.6.29)
- AMIS/JMIS का विभिन्नतहलाई आंशिकवा पूर्ण रुपमाMaintenance,OperationandManagement (MOM) को जिम्मेवारी कुनै व्यक्तिवा गैरसरकारी संस्था (NGO) लाई कबुलियत गरी दिन सकिने, (दफा1.6.37)
- AMIS/JMIS का मझौलातथा ठुला योजनाहरुको सम्पतिव्यवस्थापन योजना (AssetManagementPlan, AMP) बनाएर ज.उ.स. संग सम्झौता गरी क्रमिक रुपमा हस्तान्तरण गरिने (दफा1.6.38)
- ठुला र बहुउद्देशीययोजनाकार्यान्वयन गर्न आवश्यक स्रोत जुटाउन सिंचाइविभागले सर्वसाधारण, स्थानीयउपभोक्ता बैंक एवम् वितिय संस्थाहरुलाई कानुनीव्यवस्थाकाआधारमा सेयर जारी गर्न सक्ने (दफा1.6.43)
- अनुमतिपत्रको माध्यमबाट निजी क्षेत्रले विकास गरेको योजनामाशुल्क उठाउन सकिने । .

6. Irrigation DevelopmentVision

- IDV (2005)) ले 3 गोटा AMIS सिंचाइ प्रणालीहरुको Procedural Guidelinesबनाएर निजी क्षेत्रबाट व्यवस्थापन गराउने LongTerm (2017 सम्म) लक्ष्य राखेको छ ।

What is the factors which are dissuading private sector investor to invest in infrastructure projects? How to attract or solicitate private sector investment in road sector? What is to be done by Government and Private Sector?

"सार्वजनिक निजी साझेदारी :पूर्वाधारको बैकल्पिक लगानी"

बिषयप्रवेश :

देशको दीर्घकालीन सोंच "सम्बृद्ध नेपाल, सुखी नेपाली "पुरा गर्न, सन् २०३० सम्म दिगो बिकासको लक्ष्य हासिल गर्दै मध्यम आयस्तर भएको मुलुकमा स्तरोन्नति हुन् GDP को १३ देखि १५ %पूर्वाधार क्षेत्र त्यसमा पनि मुख्यतया यातायात क्षेत्रमा लगानी गर्नुपर्ने अवस्थामा लगानीको दृष्टिकोणबाट सरकारको ढुकुटीमा ठुलो आर्थिक व्यभार नपर्ने "OffBalanceSheet "को रुपमा परिचित सार्वजनिक निजी साझेदारी मार्फत यातायात पूर्वाधारमा लगानी लाइ सफल कार्यान्वयन गर्नुपर्ने आवश्यता छ ।

परिभाषा :

A long-term contract between a private party and a government entity, for providing a public asset or service in which the party bears, significant risk and management responsibility and remuneration is linked to performance is called private partnership.

Various Typeof PPPs

There are several different types of public-private partnership contracts, depending on various aspects such as the type of project (for example, a road or an airport), level of risk transfer, investment level and the desired outcome. Some types of PPPs include:

- Build-Own-Operate (BOO): BOO projects can be likened to the actual privatization of a facility because often there is no provision of transfer of ownership to the host government. At the end of a BOO concession agreement, the original agreement may be renegotiated for a further concession period.
- Build-Operate-Transfer (BOT): The facility is paid for by the investor but is owned by the host. The investor maintains the facility and operates during the concession period.
- Build-Own-Operate-Transfer (BOOT): Ownership of the facility rests with the constructor until the end of the concession period, at which point ownership and operating rights are transferred free of charge to the host government.
- Build-Transfer-Operate (BTO): The private sector finances a facility and, upon completion, transfers legal ownership to the public sector. The agency then leases the facility back to the private sector under a long-term lease. During the lease, the private sector operates the facility.
- Design-Build-Finance-Operate (DBFO): The private sector partner finances the project and is granted a long-term right of access of about 30 years. The DBFO partner is given specified service payments during the life of the project.

PPP सम्बन्धि मौजुदा व्यवस्था

- संबिधान मै निजि क्षेत्रलाई सहयोगीको रुपमा चित्रण गरिएको
- सार्वजनिक निजी साझेदारी तथा लगानी ऐन २०७५
- सार्वजनिक निजी साझेदारी नीति २०७२
- वैदेशिक लगानी तथा प्रविधि हस्तान्तरण ऐन २०७५
- इर्रिगेशन मास्टर plan २०१९
- पन्ध्रौ योजनामा परम्परागत सरकारी स्रोत माथिको निर्भरता घटाई लगानीका वैकल्पिक स्रोतहरु जुटाउने
- बार्षिक बजेट तथा सरकारको नीति तथा कार्यक्रम आदि

Hinderance/Hurdles/Constraints/Dissuading in 3P for Infrastructure

1. Policy Constraints

- हालको सार्वजनिक निजी साझेदारी तथा लगानी ऐन २०७५ तथा नीति २०७२ भन्दा अगाडिका कानुनी व्यवस्था अन्तर्राष्ट्रिय मान्यता अनुसार PPP का प्रावधानहरु नहुनु जस्तै Viability Gap Funding ,ProjectDevelopmentFund आदिको , TaxHoliday,जग्गा प्राप्ति सहजीकरण गर्ने बिषयबस्तु ,InternationalArbitration)UNCITRAL3 ,(P का अन्य मोडहरु DOT,MOT ,ROT ,राष्ट्र बैंकको SingleBorrowerLimit ,BiddingDocument \आदि
- नदिजन्य निर्माण सामग्रीको उपलब्धता मा कठिनाई/नदीजन्य निर्माण सामग्रीको उधोग नहुनु
- जग्गा प्राप्ति र वातावरण अध्ययन सम्बन्धि झन्झटिलो प्रक्रिया हुनु
- UnfaircompetitionbyGoNlike काठमाण्डौ हेटौडा निजीक्षेत्रलाइ दिनु तर पछि Fasttrack आफै बनाउन थाल्नु
- 3P सम्बन्धि भएको कानुनी व्यवस्थाहरु लागु गर्न उदासिता देखाउनु जस्ता RevolvingFund ,सार्वजनिक साझेदारी केन्द्र आदि
- InnovativeFinancing बारे समयानुकूल प्रवन्धहरु नगरिनु

2. InstitutionalConstraints
   - 3P मा सहयोग गर्न भनि स्थापना गरिनुपर्ने संरचनाहरु अझै नबन्नु जस्तै सा.सा .केन्द्र, सार्वजनिक निजी साझेदारी एकाई आदि
   - भएका संस्थाहरु GlobalMarket संग competition गर्ने नहुनु
   - ProjectDevelopmentFund बारे निर्णय नहुनु
   - OneWindow प्रणाली अन्तर्गत दिनुपर्ने सेवासुविधाहरु प्रदान गर्न नसक्नु
3. कार्यगत /AdministrativeConstraints
   - एकल विन्दु सेवा केन्द्रमा अधिकार सम्पन्न अन्य निकायका कर्मचारी नहुनु
   - खरिद प्रक्रिया संग सम्बन्धित ठेक्का काजगातहरु नहुनु
   - Toll सम्बन्धि व्यवस्थाहरु भविष्यमा के हुने वा सरकार पिच्छे परिवर्तन को आशंका हुनु
   - VGF/RevolvingFund/Projectpreparationfund आदिको व्यवहारिक कार्यान्वयन नहुनु
   - जोखिम व्यवस्थापन के कसरी bestway बाट गर्न सकिन्छ ?कार्यगत एकता कस्तो हुने आदि अन्यौल हुनु
   - सडक जस्तो Linearinfrastructure मा हुने uncertainties) ribbondevelopment , जग्गा अतिक्रमण ,सामाजिक पुनर्बासक मुद्दाहरु आदि (हरुको सम्बोधन नहुनु
   - LackofSufficientInvestbyNepaliInvestor) just 200 Billion(
   - DOT ,MOT ,ROT जस्ता प्राबधान लाउ गर्न मन्त्रालय, विभाग र सडक बोर्ड जस्ता संस्थाको भूमिका प्रभावकारी बन्न नसक्नु
   - बिकासका नाममा राष्ट्रियताको मुद्दाहरु उठाउनु

- पूर्वाधार अध्ययन अनुसन्धान सम्बन्धि कानुनीरुपमा व्यवस्था नहुनु आदि

4. सुधारका लागि चाल्नुपर्ने कदम

- माथि उल्लेखित विभिन्न Constraints का कारण नेपालको यातायात पूर्वाधार क्षेत्रमा 3P को अवधारणा आएको लामो समयसम्म पनि सफल भइनरहेको अवस्थामा नीति ,ऐनमा भएको व्यवस्थाहरूको धेरै समस्या समाधान गरेपनि VGF ,DisputeResolutionMechanism ,अधिकार संम्पन्न एकल विन्दु सेवा केन्द्र ,परियोजना तयारी कोष,RevolvingFund जस्ता प्रावधानहरु लाई व्यवहारिकरुपमा कार्यान्वयन योग्य रुपमा लागू गर्न ढिलाई गर्नु हुदैन ।साथै 3P को नियमावली बनाई TollchargeRegulation ,RoleofDoner,RiskSharingMechanism जस्ता सवालको उचित सम्बोधन गर्नुपर्ने देखिन्छ ।
- साथै 3Pमा सलंग्न हुने जनशक्ति को क्षमता विकास, संस्थागत विकास,RandD मा सहयोगी हुने ToolKitforTollRoad हरु समेत कार्यान्वयन गर्नुपर्ने
- 3P संग सम्बन्धित सार्वजनिक खरिद कार्यको लागि आवश्यक पर्ने कागजात/उपयुक्त विधि तथा प्राविधिक निर्देशिकाहरु तयार गरिनुपर्ने

Private Sector Participation and PPP Options in Irrigation Sector

- Government of Nepal has enacted PPP Act 2019 to provide a framework of PPP in infrastructure structure and services from both domestic and foreign private parties, particularly with the aim of attracting foreign investment. This law is an integrated version of Private Investment in Construction and Operation of Infrastructure Structure Act 2006 and Investment Board Act 2010, with provisions 10 PPP modalities (each involving transfer), including: construction-transfer, construction-operation-transfer, management-operation-transfer, maintenance/ rehabilitation-operation-transfer. It has designated the Investment Board of Nepal (IBN) as a PPP knowledge center and facilitating body for regulating PPP, though all related federal agencies (e.g., ministries) and provincial and local governments have roles in approving, regulating and promoting PPP in their respective fields according to the volume of investment required. The law does not make specific arrangements for irrigation-specific PPP, but it provides a basis on which to choose any form of PPP even in irrigation sector. However, there might be substantial legal challenges to be overcome, when project initiatives are taken.
- A number of options exist for PPP. The most commonly used contractual forms of PPP in the irrigation sector are: (i) management, performance-based and Design-Build-Operate Contracting (DBO); (ii) private sector infrastructure Design, Build, Finance and Operate (DBFO); (iii) farm (non-irrigation) service agreement; (iv) hub farm agreement; and (v) farmers' participation in the PPP contract.

- In irrigation PPP, farmers will be the recipients of irrigation services, and in some cases the farmers will be forming WUAs to support management, operation and maintenance of PPP undertaking. In order to attain sustainability, farmers' active involvement can generate the links which are missing between the public and private partners. Farmers' active participation often brings certainty to the stable demand for irrigation services.
- Various types or categories of irrigation schemes have been identified for possible inclusion in the proposed IMP. As the exact funding mechanism is still to be decided, it can be expected that financing of some large, medium and special types of projects is to involve a mix of funding through government, donor, NGO, cooperatives and farmers and hence is open up for adopting some PPP options, including private financing. Based upon the characteristics of the PPP structures and the considerations as suggested above, the PPP options for various irrigation projects of IMP have been provisionally proposed in Table.

No.	Description	Financing	Possible PPP structure
1	Large Scale IS	GoN	DBO (for 3-5 years) – and possible subsequently leasing contract
2	Medium Sized IS	GoN	DBO (for 3-5 years) – and possible subsequently leasing (LC) or management or performance based contract
3	Rehabilitation of IS	GoN	DBO (for 3-5 years) – and possible subsequently leasing (LC) or management or performance based contract
		Private	DBFO, Concession or BOOT/BOO
4	Small Scale IS	GoN	DBO (for 3-5 years) – and possible subsequently leasing (LC) or management or performance based contract
		Private	DBFO, Concession or BOOT/BOO
5	Transfer IS	GoN	DBO (for 3-5 years) – and possible subsequently a performance based contract
6	Groundwater Irrigation	GoN	DBO (for 3-5 years) – and possible subsequently leasing (LC) or management or performance based contract
7	Pumped Hill Irrigation	GoN	DBO (for 3-5 years) – and possible subsequently leasing (LC) or mgt. or performance based contract

निजी क्षेत्र संलग्न गराउन सकिने क्षेत्रहरु

- PPPModel-

Bheri-BabaiDiversion को Tunnelनिर्माण पश्चातविभिन्न क्षेत्रमाहुने लाभ र त्यसको समानुपातिकलागतलाई विभाजन गरी सो बमोजिम संरचना र पावर हाउस निजी क्षेत्रबाट बनाउन सकिन्छ ।

- ManagementContract गरेर निम्नानुसारको कार्य निजी क्षेत्रबाट गराउन सकिन्छ ।

- HeadworksandcanalRegulationwork
- RoutineMaintenanceWorks
- InstitutionalDevelopmentactivities

- Desilting
- ISFCollection

कुनै Sizable Project मा १०००० हे. भन्दा सानो वा ठुला योजनाको कुनै शाखामा सुरुवातमा परीक्षणको रुपमालिने र त्यसको सफलता र Feedback को आधार अन्यत्रलागु गर्ने ।

- ➢ पहाडी बहउद्देश्यीय योजनाको H÷W, Canal, O&MM र Power System Leaseमा क्षेत्रलाई दिन सकिन्छ ।
- आरूटर, गोर्खा
- चौरजहारी, रुकुम
- ➢ पहाडी क्षेत्रको Fall/DropमाMicro-hydropower को लागीनिजी क्षेत्रलाई अनुमतिदिएर System O&M गराउन सकिन्छ ।
- ➢ तराई को DTWलाई Domestic Water Supply मा समेत प्रयोग गर्ने गरी बहुउद्देश्यीय बनाएर Management Contract मा निजी क्षेत्र लाई संलग्न गराउन सकिन्छ ।
- ➢ Promotion of Water Sale from STW
- ➢ Medium River Multi-Purpose Project बनाएर Management Contract मा निजाक्षेत्रलाई संलग्न गराउन सकिन्छ ।

3 Planning and Design

CANAL HEADWORKS

GENERAL

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 offtaking main canal are termed “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 offtaking channel. The water level in the source channel is raised to the required level so as to divert the required supplies into the offtaking channel. The diversion headworks should be capable of regulating the supplies into the offtaking channel. If required, it should be possible to divert all the supplies (at times of keen demand and low supplies) into the offtaking channel. The headworks must have an arrangement for controlling the sediment entry into the channel offtaking from a river. By raising the water level, the need of excavation in the head reaches of the offtaking channel is reduced and the command 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.

Most headworks in Nepal are diversion headworks which can be either temporary or permanent. For temporary diversion headworks, bunds are constructed every year across the source river after floods. Sometimes, temporary diversion headworks are constructed in the beginning and when the demand for irrigation has developed sufficiently, they are replaced by permanent headworks. The Upper Ganga canal was run with temporary headworks for about sixty years before the permanent headworks were constructed. In order to further increase the irrigation capacity of the Upper Ganga canal system, these permanent headworks have now been replaced by new structures. For all important headworks, only permanent headworks should be constructed.

LOCATION OF HEADWORKS ON RIVERS

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 siting headworks. Usually, the command area is away from the hilly stage, and it would, therefore, involve avoidable expenditure to construct a channel from headworks located in the hilly stage to its command area. In the delta stage, the irrigation requirements are generally less and also the nature of the river at this stage poses other problems.

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 command area. If both stages are equally suitable for siting the headworks from command 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.

The initial cost of headworks in the boulder stage is generally smaller than that in the alluvial stage because of: (a) local availability of stones, (b) smaller width of river (requiring smaller length of weir), (c) smaller scour depths which reduce the require- ments of cutoffs and other protection works, and (d) close proximity of higher banks which requires less extensive training works.

An irrigation canal offtaking from a river in the boulder region will have a number of falls which may be utilised for generation of electricity. There is almost no scope for the generation of electricity in this manner in the alluvial reach of a river.

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 ex- penditures incurred on the headworks. Construction of temporary bunds is gener-ally not possible in the alluvial stage of the river.

An irrigation channel offtaking 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 offtaking channel. In alluvial reach of the river this loss of water is much less.

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 offtaking 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.

Once the stage of a river has been chosen for locating the headworks, the site of the headworks is selected based on consideration of its suitability for the barrage (or weir), the undersluices, and the canal head regulator.

For irrigation purposes, the site for headworks should result in a suitable canal alignment capable of serving its command area without much excavation. For siting the headworks,

theriver 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 thenodal point.
From sediment considerations, the offtaking channel should be located at the downstreamend 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 offtaking 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 centre 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.

DIFFERENT UNITS OF HEADWORKS

Diversion headworks mainly consist of a weir (or barrage) and a canal head regulator. A weir has a deep pocket of undersluice portion upstream of itself and in front of the canal head regulator on one or both sides. The undersluice bays are separated from other weir baysby means of a divide wall. In addition, river training structures on the upstream and downstream of weir, and sediment excluding devices near the canal head regulator are provided. Detailedmodel investigations are desirable to decide the location and layout of headworks and its component units. A typical layout of diversion head works is shown in Fig. 13.1

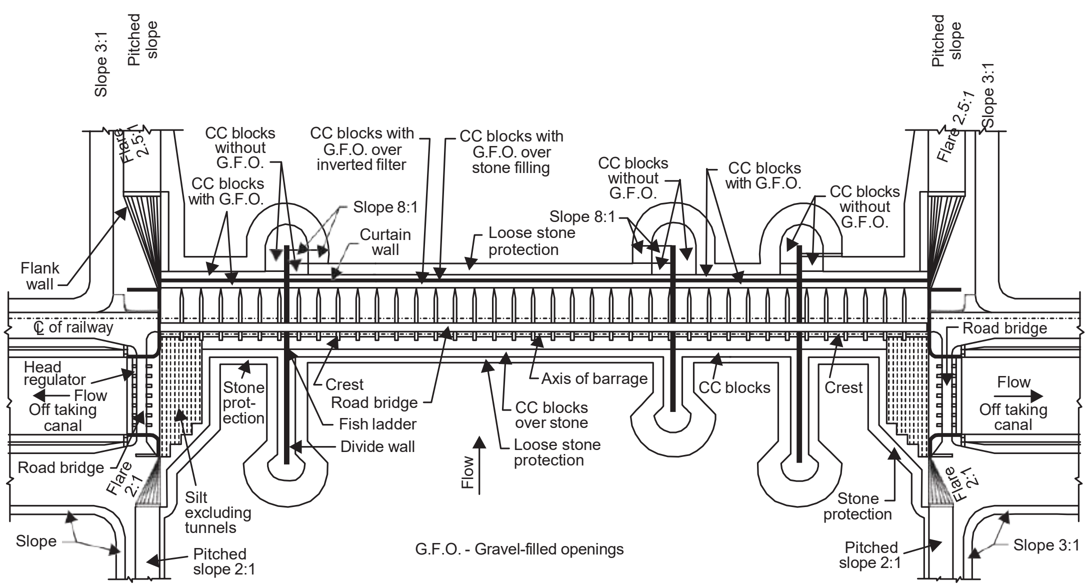


Fig. 13.1 Typical layout of headworks (1)

WEIR (OR BARRAGE)

A weir is an ungated barrier across a river to raise the water level in the river. It raises the water level in the river and diverts the water into the offtaking canal situated on one or bothof the river banks just upstream of the weir. Weirs are usually aligned at right angles to the direction of flow in the river. Such weirs will have minimum length and normal uniform flowthrough all the weir bays thereby minimising the chances of shoal formation and oblique flow.

To increase the water level, the weir crest is raised above the river bed. Part of the raising of the water level is obtained by shutters provided at the top of the weir crest. These shutters are dropped down during floods so that the afflux is minimum. The afflux is defined as the difference in water level between the upstream and downstream of a structure under free flow conditions as a result of construction of the structure across a river. Controlling pond levels by means of shutters becomes difficult when the difference between the pond level and the crest level is higher than 2.0 m. In such cases, a gate-controlled weir, better known as barrage, is preferred. Barrage is a gate-controlled weir with its crest at a lower level. A barrage and weir are similar structures and differ only in a qualitative sense. The crest of a barrage is usually at a lower level and the ponding up of the river for diversion into the offtaking canal is achieved by means of gates (instead of shutters). Barrages are considered better than weirs due to the following reasons:

Barrages offer better control on the river outflow as well as discharge in the off taking canal. With proper regulation and with the help of undersluices and sediment excluders, the upstream region in the vicinity of the headworks can be kept free of sediment deposition so that sediment-free water enters the offtaking canal. Because of the lower crest level of a barrage, the afflux during floods is small. It is possible to provide a roadway across the river at a relatively small additional cost.

Because of these advantages, barrages are usually constructed at the site of headworks on all important rivers. At some barrages, the raised crest may not be provided at all and the complete ponding is obtained only by means of gates.

The procedure of design of a barrage is similar to that of a weir. Weirs are of the following three types:

Masonry weirs with vertical downstream face,
Rockfill weirs with sloping apron, and
Concrete weirs with glacis.

Masonry Weirs with Vertical Downstream Face

Figure 13.3 shows the typical sketch of a masonry weir which consists of a horizontal masonry floor and a masonry crest with vertical (or nearly vertical) downstream face. Shutters provided at the top of the crest raise the water level further. During floods, these shutters are dropped down to pass the floods effectively and reduce the afflux upstream. The stability of the crest should be examined for the following conditions:

The water level on the upstream side is up to the top of the shutters with no flow on the downstream side and all the water is diverted into the offtaking canal. The overturning moment caused by the water pressure on the upstream side must be resisted by the weight of the crest without any tension at its upstream end. The stability of the crest against sliding due to water pressure should also be examined.

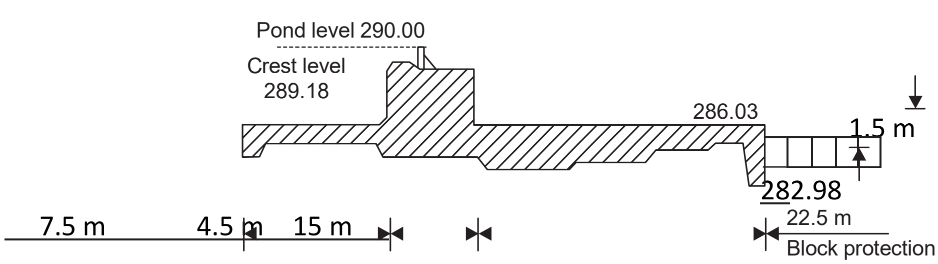


Fig. 13.3 The Bhimgoda weir (old) on the Ganga river at Hardwar

When the shutters are dropped down, water flows over the crest and the overturning moment is reduced due to the lowered water level on the upstream and presence of water on both sides of the crest. However, there will be some loss of weight (and, hence, the resisting moment) of the crest due to floatation because of the crest not being completely impervious. It is impossible to determine the amount of this loss of weight accurately. The reduced resisting moment is calculated on the basis of full weight of the masonry above the downstream level and submerged weight below the downstream level. The safety of the crest is examined for different stages of dis-charge up to the maximum flood discharge. At all such stages, the resisting moment must be more than the overturning moment and there should be no tension at the upstream end of the crest.

Rockfill Weirs with Sloping Aprons

Figure 13.4 shows the longitudinal section of a typical rockfill weir whose main body consists of dry boulders packed in the form of glacis with few intervening walls. This type of weir is the simplest one, but requires a large quantity of stones for construction as well as maintenance. As such, this type of weir is suitable in areas where a large quantity of stones is available in the vicinity of the site and where labour is cheap.

H.F.L. 203.10 Pond level 201.67 H.F.L. 202.51:20

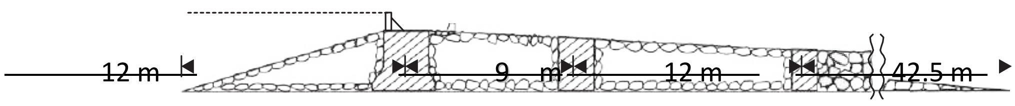


Fig. 13.4 The Okhla weir (old) on the Yamuna river near Delhi

Concrete Weirs with Glacis

Figure 13.5 shows the longitudinal section of a typical concrete weir in which the excess energy of overflowing water is dissipated by means of a hydraulic jump which forms near the downstream end of the glacis. Barrages are also constructed like concrete weirs. Design of such weirs is mainly based on the method proposed by Khosla et al. discussed in Chapter 9. On pervious foundations, only concrete weirs are constructed these days. Their detailed design requires the knowledge of: (i) the maximum flood discharge and corresponding level of the river at and near the selected site for weir, (ii) the stage-discharge curve of the river at the weir site, and (iii) the cross-section of the river at the weir site.

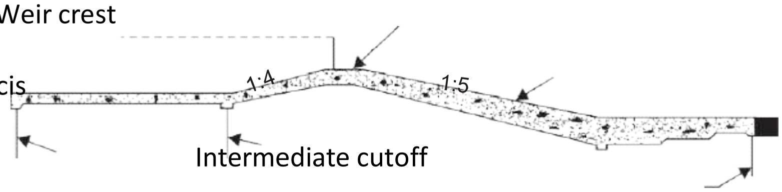


Fig. 13.5 Concrete weir

Based on the site conditions, general and economic considerations, and other data, the designer decides (i) the afflux, (ii) the pond level, (iii) the minimum waterway (or the maximum discharge per metre length of weir), and (v) the weir crest level.

UNDERSLUICES

The construction of weir across a river results in ponding up of water and causes considerable sediment deposition just upstream of the canal head regulator. This sediment must be flushed downstream of the weir. This is done by means of undersluices (also called sluice ways or scouring sluices). A weir generally requires deep pockets of undersluices in front of the head regulator of the offtaking canal, and long divide wall to separate the remaining weir bays from the undersluices. The undersluices are the gate-controlled openings in continuation of the weir with their crests at a level lower than the level of the weir crest. The undersluices are located on the same side as the offtaking canal. If there are two canals each of which offtakes from one of the banks of the river, undersluices are provided at both ends of the weir.

The undersluices help in keeping the approach channel of the canal head regulators relatively free from deposition of sediment, and minimise the effect of the main river current on the flow over the head regulator. In addition, the undersluices are also useful for passing low floods, after meeting the requirements of the offtaking canal, without having to raise the gates or drop the weir shutters. With the provision of the undersluices, the weir shutters have to be dropped (or the gates have to be raised) only to pass the high floods which occur only during the monsoon. The shutters are again raised (or the gates are lowered) at the end of the monsoon.

The level of the crest of an undersluice is related to: (i) cold weather river bed level at the site of the weir, and (ii) the crest level of the head regulator. The crest of the undersluices generally coincides with the lowest cold weather level of the river bed at the site of the weir. Also, the crest of the undersluices should be kept at least 1.20 m below that of the head regulator so that the sediment deposited upstream of the undersluices (and the head regulator) does not enter the offtaking canal and can be carried to the downstream of the undersluices. If a sediment excluder (a structure to reduce entry of sediment into the canal) is to be provided, it becomes necessary to lower the crest of the undersluices to about 2.0 m below the crest of the head regulator. Alternatively, the crest of the regulator is raised.

The discharge capacity of the undersluices is kept equal to the maximum of the values given by the following three considerations:

To ensure sufficient scouring capacity, the capacity of the undersluices should be at least twice the canal discharge.

To reduce the length of the weir, the undersluices should be capable of passing about 10 to 20 per cent of the maximum flood discharge at high floods.

The undersluice should possess enough capacity to pass off low floods with the water surface in the reservoir at pond level so that the need of lowering the weir shutters (or raising the weir gates) does not arise.

The longitudinal section of the undersluices will be similar to that of a weir and, hence, the design procedure is the same as that of a concrete weir. However, because of the larger discharge per unit length, the undersluices would need much heavier protection downstream. When sediment excluders are provided, the width of undersluices is determined by the velocity required to induce siltation. On all major headworks, the design of undersluices and divide wall is finalised on the basis of model investigations.

AFFLUX, WATERWAY, AND DIFFERENT LEVELS FOR WEIRCONSTRUCTION

Due to the construction of a weir across a river, the high flood level of the river upstream of the weir rises. This rise is termed the afflux and is usually represented as the difference in the total energy levels of the upstream and downstream of the weir. In the beginning, the afflux is confined to a short reach of the river but, extends gradually very far upstream in case of the alluvial rivers. The top level of guide banks and marginal bunds and also the length of the marginal bunds are decided by the amount of the afflux. Besides, the afflux affects the dynamic action downstream of the weir and also the location and parameters of the hydraulic jump. A higher afflux may reduce the width of the waterway but, increases the discharge per unit length of weir. This results in increased depth of scour which, in turn, increases the cost of protection works. A higher afflux also increases the risk of failure of river training structures due to possible outflanking.

In the case of weirs located on alluvial rivers, an afflux of 1 m is considered satisfactory in the upper and middle reaches of the river. In the lower reaches with flat gradients, the afflux should be limited to about 0.3 m.

The pond level is the water level which must be maintained in the undersluices pocket (i.e., upstream of the canal head regulator) so that full supply level can be maintained in the canal when full supply discharge is fed into it. The full supply level of a canal at its head is obtained from the longitudinal section of the canal. The pond level is kept about 1.0 to 1.2 m higher than the full supply level of the canal so that sufficient working head is available even when the head reach of the canal has silted up, or when the canal has to be fed excess water. If under certain situations, there is a limitation of pond level, the full supply level is fixed by subtracting the working head from the pond level.

The waterway and the afflux are interdependent. Normally, in plains, the width of waterway is kept equal to about 10 to 20% more than Lacey's regime perimeter for the design flood discharge. In rivers with coarser bed material, the width of waterway can be kept about 10-20% smaller than Lacey's perimeter. A smaller waterway increases the afflux and the cost of protection works. On the other hand, a larger waterway is uneconomical and may cause oblique approach, thereby silting part of the waterway. The ratio of the overall length of the weir provided to the minimum stable width of river obtained from Lacey's equation for the design flood discharge is termed looseness factor

The weir crest level, afflux, waterway, and pond level are interrelated and a suitable set of values of all these four parameters are decided within their respective limits. The pond level can be maintained by keeping the weir crest at the pond level. Alternatively, the weir crest can be kept at a lower level and the pond level is, then, maintained with the help of shutters or gates. The level of the weir crest is decided as follows:

From the stage-discharge curve at the weir site, the high flood level (HFL) for the design flood (usually 50- to 100-year frequency flood) discharge is determined.

The level of the downstream total energy line (TEL) is determined adding the veloc- ity head to the HFL. The velocity of flow is computed using the Lacey's regime equa- tion for velocity.

The permissible afflux is added to the level of the downstream TEL to obtain the level of the upstream TEL.

The discharge intensity q is determined by dividing the design flood discharge by the width of clear waterway.

Using the relation,

$q = Ck^{3/2}$ (13.1)

The height of TEL, above the weir crest, k is determined. Here, k (in meters) is the total head with respect to the weir crest. In Equation, the value of C depends on many factors, such as the head over the weir crest, shape and width of the crest, thecrest height over the upstream floor, and the roughness of the crest surface. It is, therefore, advisable to estimate C by the use of model studies if the values based onprototype observations based on similar structures are not available. For broad- crested weirs having crest width more than 2.5 times the head over the crest, C maybe taken as 1.71. For the crest whose width is less than 2.5 times the head over thecrest, the value of C is taken as 1.84.

The level of the weir crest is obtained by subtracing k from the level of the upstreamTEL.

After fixing the weir crest level, length and suitable number of weir bays are de- cided. The total discharge capacity of the weir and undersluice bays is worked out using the following discharge equation which takes into consideration the reduction in width of flow on account of end contractions.

$Q = C (L - KnH) H^{3/2}$ (13.2)

Here, L is the overall waterway, H the head over the crest, n the number of end contractions, and K is a coefficient which ranges from 0.01 to 0.1 depending upon the shape of the abutment and the pier nose. The exact value of the coefficient C depends on several factors, viz., head over the crest, the shape and size of the crest, the height of the crest over the upstream floor, and the roughness of the crest surface. Use of model studies is suggested for the estimation of C. In the absence of such studies, C can be assumed as 1.71 in SI units.

The height of shutters or gates will be equal to the difference between the pond level and the level of the weir crest.

For weirs without shutters (or gates), the crest level should, obviously, be at the required pond level. For weirs with falling shutters, the crest level should not be lower than 2 m below the pond level as the maximum height of the falling shutters is normally limited to 2m. If the crest level so fixed causes too much of afflux, the waterway of weir may be suitably increased. For barrages too, the crest level is similarly determined by the head required to pass the design flood at the desired afflux. It is desirable that the crest (of the barrage) and the upstream floor levels of the undersluices be kept at the lowest bed level of the deep channel of the river as far as practicable. The upstream floor level of the remaining bays should be kept normally 0.5 to 1.0 m above the upstream floor level of the undersluice bays or the general river bed level.

As a result of the construction of weir across a river, the downstream bed levels will be lowered due to degradation (or retrogression) and, hence, the downstream HFL will also be lowered. The lowering of water levels due to retrogression on the downstream increases the exit gradient. Retrogression is relatively more in alluvial rivers carrying fine sediment and having steep slope. When the proposed weir is sited downstream of a dam, the retrogression increases. Retrogression should always be considered for the design of the downstream floor and the downstream protection works.

During high floods, the river water carries lot of sediment which reduces the extent of retrogression and, hence, lowering of HFL is only marginal - of the order of about 0.3 to 0.5 m. But, during low floods the river water downstream of the weir is relatively clear and may increase retrogression thereby lowering the HFL by an appreciable amount ranging from 1.25 to 2.25 m, depending upon the amount of sediment in the river bed and the bed slope of the river (2). At the design flood in an alluvial river, the reduction in river stages due to retrogression may be considered to vary from 0.3 to 0.5 m depending upon whether the river is shallow or confined during floods (2). For other discharges, the effect of retrogression may be obtained by the variation of retrogressed flood levels with the flood discharges. The downstream TEL is lower than the upstream TEL by an amount H_L which is equal to the sum of the assumedafflux and the estimated retrogression.

During the first few years of weir construction, the sediment-carrying capacity of the river decreases due to ponding up of water upstream of the weir. This results in continuous deposition of the sediment upstream of the weir and the bed level rises. Ultimately, the bed slope regains its original slope and the afflux extends still further upstream of the backwater profile. The marginal bunds, constructed to take care of the rise in water level in the backwater region, will, then, have to be extended further upstream. A stage is thus reached when the upstream pond takes no more sediment. Since the offtaking canal withdraws relatively sediment-free water, the sediment is now carried downstream of the weir with reduced water discharge (and, hence, reduced sediment-carrying capacity). Therefore, the sediment will start depositing on the downstream side and raise the bed levels lowered due to initial degradation. Sometimes, the bed levels can rise even beyond the original bed levels.

DESIGN OF WEIR

Dimensions of different parts (such as vertical cutoff, impervious floor, etc.) of a weir on a permeable foundation are determined from surface and subsurface flow considerations as discussed in the following sections.

The data required for this purpose include:

longitudinal section and cross-section of river at the weir site,

stage-discharge relationship including HFL and the corresponding discharge,

characteristics of sediment and river bed material, and

canal data such as full supply level and the corresponding discharge, canal cross-section, longitudinal section, etc.

Vertical Cutoffs

Vertical cutoff (or sheet pile lines) at the upstream and downstream ends of a weir are always provided to guard against scouring at the upstream and downstream ends and the piping effects at the downstream end. Intermediate cutoffs, provided at the ends of the upstream and/ or the downstream slopes of the impervious floor, are useful in holding the main structure, i.e., the weir, in case of failure of the upstream and/or the downstream cutoffs. The depth of cutoffs should be such that its bottom is lower than the level of possible flood scour at that section. The downstream cutoff, in addition, should also be sufficient to reduce the exit gradient within safe limits which is decided by the subsurface conditions. The depth of scour below HFL, R is given by Lacey's equation, Eq. (8.33),

$$R = 1.35\left(\frac{q^2}{f}\right)^{1/3} \qquad (13.3)$$

For fixing the depths of cutoffs, the scour depth R should be calculated with discharge intensity q taking into account the concentration factor. The concentration factor is a factor by which the discharge per unit length of a weir, assuming uniform distribution across the river width, is required to be multiplied to obtain the design discharge per unit length for designing stilling basin and the cutoffs of the weir (2). The concentration factor accounts for the nonuniform distribution of the flow along the waterway during the operation of weir bays.

Equation (13.3) computes scour depth R for the regime condition of flow existing during high floods. Other features, such as bends, increase the depth of scour. The extent of scour (below water level) in a river with erodible bed material varies at different places along a weir as shown in Table 13.2.

Table 13.2 Likely extent of scour along a weir (2)

Location	Range	Mean
Upstream of impervious floor	1.25 R to 1.75 R	1.50 R
Downstream of impervious floor	1.75 R to 2.25 R	2.00 R
Noses of guide banks and divide wall	2.00 R to 2.50 R	2.25 R
Transition from nose to straight part	1.25 R to 1.75 R	1.50 R
Straight reaches of guide banks	1.00 R to 1.50 R	1.25 R

If the sub-stratum contains any continuous layer of clay in the vicinity of the downstream cutoff, the depth of the upstream and downstream cutoffs should be suitably adjusted to avoidincrease of pressure under the floor.

Weir Crest, Glacis, and Impervious Floor

The weir crest is provided flat at the computed level with a width of about 2 m (2). If the weir is to behave as a broad-crested weir, the width should be more than 2.5 times the head over the weir. The upstream slope of the weir is fixed between 2(H):1(V) to 3(H):1(V).

The downstream slope of the weir crest and downstream horizontal floor (i.e., stilling basin) should be such that they result in the maximum dissipation of energy through stable hydraulic jump besides being economic. The slope of the downstream glacis should be around 3(H):1(V). The level of the downstream horizontal floor is fixed in such a manner that the hydraulic jump starts at the end of the glacis or upstream for all discharges. The location of the hydraulic jump is determined (for high flood and pond level discharges) by using the method of Sec. 9.2.8. The floor level is kept at or below the lower of the required floor levels for these two conditions. A concentration factor of 1.2 is usually adequate for the design of the stilling basin.

The total floor length of impervious floor includes the downstream basin length, glacis, weir crest, and upstream floor. The impervious floor in conjunction with the downstream cutoff

should result in safe exit gradient. Besides, the hydraulic jump must remain confined within the downstream floor. It should also satisfy the requirements of uplift pressures.
The length of the downstream horizontal floor should be such that the entire jump is confined only to the floor. This will ensure that the filter and the stone protection provided on the downstream of the floor are not affected adversely by the jump. Hence, the length of the downstream horizontal floor is kept equal to the length of the jump which is equal to five to six times the height of the jump [(i.e., 5 to 6 $(h_2 - h_1)$)]. Here, h_1 and h_2 are pre-jump and post-jump depths of flow. Obviously, the maximum height of the jump should be considered for these calculations.
The length of the upstream horizontal floor, provided at the river bed level, is decided in such a manner that the resulting exit gradient, G_E is less than the safe exit gradient for the soil under consideration. In Eq. (9.66), the depth of the downstream cutoff is measured below the bed (or the scoured bed, if scour is anticipated). Theoretically, the safe exit gradient should be equal to the critical gradient which is unity for average soils as given by Eq. (9.50). However, in practice, the safe exit gradient is kept lower than the critical gradient mainly due to non-uniformity of the soil conditions. Besides, the presence of faults and fissures in the subsoil, possibility of scouring up to the bottom of the vertical cutoff, sudden changes in head due to sudden dropping of gates, and similar factors suggest that the safe exit gradient be kept lower than the critical gradient. The recommended values of the safe exit gradient are 1/4 to 1/5 for shingles, 1/5 to 1/6 for coarse sand, and 1/6 to 1/7 for fine sand (2).
Thus, knowing the value of the safe exit gradient G_E, the head H, and the depth of the downstream cutoff d, one can determine the total floor length required, i.e., b, using Eq. (9.66)which is

$$G_E = \frac{H}{d}\frac{1}{\pi\sqrt{\lambda}} \qquad (13.4)$$

where, $\lambda = \frac{1}{2}\left[1 + \sqrt{1 + (b/d)^2}\right]$

The thickness of the impervious floor is decided from considerations of the uplift pressures which can be determined using the method proposed by Khosla et al. and as explained in Chapter 9. The thickness of floor in the jump trough should be examined for different stages of flow. For other parts of the downstream floor, however, the maximum uplift would occur when the water is at the pond level on the upstream without any tail-water (or flow) on the downstreamside.

Upstream and Downstream Loose Protection

To protect a weir structure from the adverse effects of scour at the upstream and downstream ends of the structure, some additional measures in the form of cement concrete blocks over loose stones are needed.
Just beyond the upstream end of the impervious floor, pervious protection in the form of cement concrete blocks is provided. The blocks should be of adequate size so as not to get dislodged. For barrages in alluvial rivers, the blocks of size 1500 mm × 1500 mm × 900 mm are used (2). The size of the blocks should, obviously, be larger in case of barrages in the boulder reach of a river. The length of the upstream block protection is kept approximately equal to the depth of scour below the floor level. The blocks are laid over loose stones.
The pervious block protection is provided beyond the downstream end of the impervious floor as well. The cement concrete blocks of size not less than 1500 mm × 1500 mm × 900 mm

are laid over a suitably designed inverted filter. The space of about 75 mm between the adjacent blocks is packed with gravel. The length of the downstream block protection is kept approximately equal to 1.5 times the scour depth below the floor level.

The graded inverted filter should approximately conform to the following criteria (2):

$$\frac{D_{15} \text{ of filter}}{D_{15} \text{ of river bed material}} \geq 4 \geq \frac{D_{15} \text{ of filter}}{D_{85} \text{ of river bed material}}$$

The filter may be provided in two or more layers. The grain size curves of the filter material and the bed material should be approximately parallel.

A toe wall of masonry or concrete should also be provided at the end of the inverted filter to prevent the filter from getting disturbed. The wall should extend up to 500 mm below the bottom of the filter.

Beyond the block protection on the upstream and downstream of a weir on permeable foundation, launching aprons of loose boulders or stones are provided so that these stones can spread uniformly over the scoured slopes. These stones should not weigh less than 400 N, and should always be larger than 300 mm in size. In case of non-availability of suitable material for launching aprons, cement concrete blocks or stones packed in wire crates are used. The quantity of stone provided in launching aprons should be adequate to cover the slope of scour holes (ranging from 2 (H) : 1 (V) to 3 (H) : 1 (V) for alluvial rivers and 1.5 (H) : 1 (V) for boulder rivers) with a thickness of 1.25 T where T (i.e., thickness of pitching) is to be obtained from Table 13.3.

Table 13.3 Thickness of pitching (T) for loose stone protection in millimetres (2)

Type of river bed material	River slope (m/km)				
	0.05	0.15	0.20	0.30	0.40
Very coarse	400	500	550	650	700
Coarse	550	650	700	800	850
Medium	700	800	850	950	1100
Fine	850	950	1000	1100	1150
Very fine	1000	1100	1150	1250	1300

The depth of scour holes, calculated from Lacey's equation, is to be modified according to Table 13.2 for different locations of launching aprons. No allowance in the form of concentration factor need be made for computing normal scour depth for the upstream and downstream protection works (2).

The total quantity of stone for launching apron, worked out as mentioned above, should be laid in a length of about 1.5 D to 2.5 D. Here, D is the depth of scour below the floor level. The higher values are to be used for flatter launched slope. The thickness of loose stone at the inner edge should correspond to the required quantity of stone for thickness of launched apron equal to T. The material required for extra thickness of 0.25 T on launched slope should be distributed over the length of apron in the form of a wedge with increasing thickness towards the outer edge.

Example 13.1 For the following data related to a canal headworks, design the profile of the weir section:

Flood discharge	= 3000 m^3/s
Maximum winter flood discharge	= 300 m^3/s
HFL before construction of weir	= 255.5 m
RL of river bed	= 249.5 m
Pond level	= 254.5 m
Lacey's silt factor (for the river bed material)	= 0.9
Permissible afflux	= 1.0 m
Bed retrogression	= 0.5 m
Offtaking canal discharge	= 200 m^3/s
Looseness factor	= 1.1
Concentration factor	= 1.2
Permissible exit gradient	= 1/7

The stage-discharge curve of the river is as shown in Fig. 13.6.

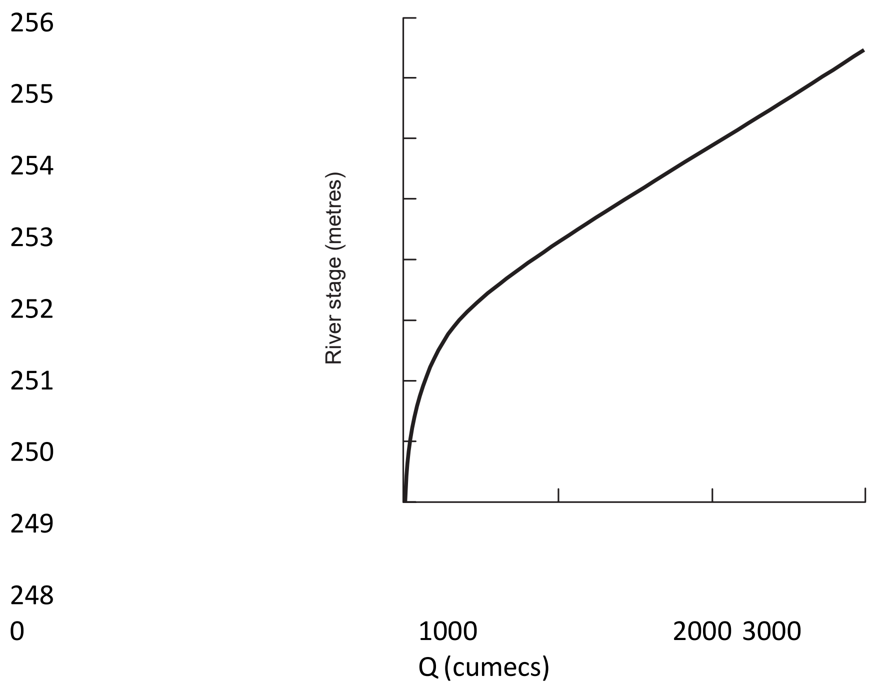


Fig. 13.6 Stage-discharge curve for Example 13.1

Solution:

Lecey's waterway, $P = 4.75\sqrt{Q} = 4.75\sqrt{3000} = 260.17$ m

Actual width of waterways = P × Losseness factor

= 260.17 × 1.1 = 286.19 m

Lacey's regime velocity $= \left(\dfrac{Qf^2}{140}\right)^{1/6} = \left(\dfrac{3000 \times 0.9 \times 0.9}{140}\right)^{1/6}$

$= 1.61$ m/s

Velocity head $= \dfrac{1.61 \times 1.61}{2 \times 9.81} = 0.13$ m

Level of d/s TEL = 255.5 + 0.13

= 255.63 m (255.13 m after retrogression)Level of u/s TEL = d/s TEL + Afflux = 255.63 + 1.0

= 256.63 m (256.13 m after retrogression)

Level of u/s HFL = 256.63 – 0.13 = 256.5 m (256.0 m after retrogression)

The undersluice bays should carry maximum of the following three discharges:

Twice the discharge of the offtaking canal = 2 × 200 = 400 m^3/s

20% of the design flood discharge = 0.2 × 3000 = 600 m^3/s

Maximum winter flood discharge = 300 m^3/s

$\therefore$ Undersluice discharge = 600 m^3/s

Providing the undersluice crest at the river bed level (i.e., 249.5 m), the total head on the undersluice crest during high flood condition = 256.63 – 249.5 = 7.13 m.

Discharge intensity q through the undersluice portion = 1.71 $(7.13)^{3/2}$ = 32.56 $m^3/s/m$. On providing two bays of 10.0 m each (and assuming pier contraction coefficient = 0.1), Discharging capacity of undersluices = 1.71 (20 – 0.1 × 2 × 1 × 7.13) $(7.13)^{3/2}$

= 604.77 m^3/s = 604 m^3/s (say)

It should be noted that the pier contraction coefficient would depend on the shape of the pier and may be as small as 0.01 for rounded nose piers.

The weir portion has to carry the remaining flood discharge

= 3000 – 604 = 2396 m^3/s

On providing 18 bays of 15 m each for the weir portion,

Discharge intensity for weir $= \dfrac{2396}{18 \times 15} = 8.87\ m^3/s/m$

Height of TEL over the weir crest $= \left(\dfrac{8.87}{1.84}\right)^{2/3} = 2.85$ m

Here, the formula q = 1.84 $k^{3/2}$ has been used since the weir crest is usually not wide enough to behave as a broad crest.

Required level of the weir crest = 256.63 – 2.85 = 253.78 m

Provide weir crest at 253.70 m so that the total discharge through the weir andundersluice

= 1.84 (270 – 0.1 × 17 × 2 × 2.93) $(2.93)^{3/2}$ + 604

= 2401 + 604

= 3005 m^3/s against the required value of 3000 m^3/s

and clear waterway = 270 + 20

= 290 m against the required value of 286.19 m

Thus, the requirements of discharge capacity and clear waterway are met by providing 18 weir bays (with weir crest at 253.70 m) of 15 m each and two undersluice bays (with crest at 249.5 m) of 10 m each.

Required height of undersluice gates = 254.5 – 249.5 = 5.0 m

Required height of weir shutters = 254.5 – 253.7 = 0.8 m

Hydraulic jump calculations for weir sections:

Quantity	High flood condition	Pond level condition
Discharge intensity, q (in m^3/s/m)	$1.84\,(2.93)^{3/2} \times 1.2 = 11.07^{a}$	$1.84\,(0.8)^{3/2} \times 1.2 = 1.58^{a}$
Critical depth, h_c	2.32	0.63
D/S flood level (after retrogression) (m)	255.00	251.25*
D/S TEL (m)	255.13	251.33*
U/S TEL (m)	257.01***	254.58**
Head loss, H_L (m)	1.88	3.25
H_L/h_c	0.81	5.16
E_2/h_c (from Table 9.1)	2.01	2.74
E_2 (m)	4.66	1.73
$E_1 = E_2 + H_L$ (m)	6.54	4.98
Pre-jump depth, h_1(m)	1.07	0.162
Post-jump depth, h_2 (m)	4.32	1.69
Height of the jump = $h_2 - h_1$ (m)	3.25	1.528
Length of concrete floor required (= 5 × height of the jump) (m)	16.25	7.64
Level of the jump = D/S TEL – E_2 (m)	250.47	249.60

The level of the downstream floor should be at or lower than 249.60 m and the length of the downstream horizontal floor should be equal to or more than 16.25 m. However, the river bed is at 249.0 m. Therefore, provide downstream horizontal floor of length 17 m at 249.5 m.

Upstream and Downstream Cutoffs:

Concentration factor is accounted for in the computations of scour depth for determination of depths of cutoffs. Using Eq. (13.3), the depth of scour below HFL,

$$R = 1.35\left(\frac{11.07 \times 11.07}{0.9}\right)^{1/3} = 6.95 \text{ m}$$

---

[a] after taking into account concentration factor of 1.2.

* At the pond level, total discharge through weir and undersluice bays

$= 1.84\,(270 - 0.1 \times 2 \times 17 \times 0.8)\,(0.8)^{3/2} + 1.71\,(20 - 0.1 \times 2 \times 1 \times 5)\,(5)^{3/2}$

$= 715.15\ m^3/s$

$\therefore$ Corresponding river stage = 251.75 m

$\therefore$ D/S flood level (after retrogression) = 251.25 m

For Q = 715.15 m^3/s,

Lacey's regime velocity = 1.267 m/s

$\therefore$ Velocity head = 0.08 m

Level of D/S TEL = 251.25 + 0.08 = 251.33 m

** Level of U/S TEL = 254.50 + 0.08 = 254.58 m

*** Total head over the crest (for q = 11.07 $m^3/s/m$) = $(11.07/1.84)^{3/2}$ = 3.31 m Level of U/S TEL = 253.7 + 3.31 = 257.01 m

RL of bottom of the scour hole on the upstream side
= Upstream HFL – 1.5R
= 256.5 – 1.5 × 6.95 = 246.08 m

Therefore, provide upstream cutoff up to the elevation of 246.0 m. Further, RL of bottom of the scour hole on the downstream side
= downstream HFL – 2R
= 255.5 – 2 × 6.95 = 241.6 m

Therefore, provide downstream cutoff up to the elevation of, say, 241.5 m.

Then, depth of the downstream cutoff, d = 249.5 – 241.5 = 8.0 m

(The permissible exit gradient of 1/7 indicates absence of boulder material in the bed and one may, therefore, provide sheet piles instead of concrete cutoff.)

Maximum static head = Pond level – D/S floor level = 254.5 – 249.5 = 5.0 m

Using Eq. (13.4), the exit gradient, $G_E = \frac{1}{7} = \frac{5.0}{8.0}\frac{1}{\pi\sqrt{\lambda}}$

$\Rightarrow \quad \lambda = 1.94$

Further, $\quad \lambda = 0.5\left[1 + \sqrt{1 + (b/d)^2}\right] = 1.94$

$\Rightarrow \quad b = 21.61$ m

Total impervious floor length:

Downstream horizontal floor = 17 m

Horizontal length of the downstream glacis with a slope of 1(V) : 3(H)
= 3 (253.70 – 249.5) = 12.6 m

Let the crest width be 2 m.

Horizontal length of the upstream slope (1(V):2(H)) of the weir
= (253.7 – 249.5) × 2 = 8.4 m

After providing these essential floor lengths, the total imervious floor is already 40 m long. Providing 2.5 m long upstream horizontal floor, the total length of the impervious floor is 42.5 m, which is more than the required value of 21.61 m from the exit gradient consideration.

Loose protection on upstream and downstream:

Concentration factor is not to be accounted for in computation of scour depth for designing loose protection.

Discharge intensity of weir at high flood = 1.84 $(2.93)^{3/2}$ = 9.23 $m^3/s/m$

Depth of scour below HFL, $R = 1.35\left(\frac{9.23 \times 9.23}{0.9}\right)^{1/3} = 6.15$ m

$\therefore$ RL of bottom of scour hole on the upstream side
= upstream HFL – 1.5R
= 256.5 – 1.5 × 6.15 = 247.28 m

Depth of scour below river bed = 249.5 – 247.28 = 2.22 m

Similarly, the depth of scour below the river bed on the downstream
= 249.5 – (downstream HFL – 2R)
= 249.5 – (255.5 – 2 × 6.15) = 249.5 – 243.2 = 6.3 m
Thus,
Length of upstream block protection ≈ 2.22 m (Provide 2.3 m) Length of upstream launching apron = 2 × 2.22 = 4.5 m (say)
Length of the downstream block protection ≈ 1.5 × 6.3 m (Provide 10 m)
Length of downstream launching apron = 1.5 × 6.3 = 10 m (say)
The line sketch of the designed weir profile is shown in Fig. 13.7. Following the procedure of Example 10.2, one can determine the floor thickness at various locations of the weir section. Similarly, one can determine the profile and design the floor thickness of the undersluice portion.

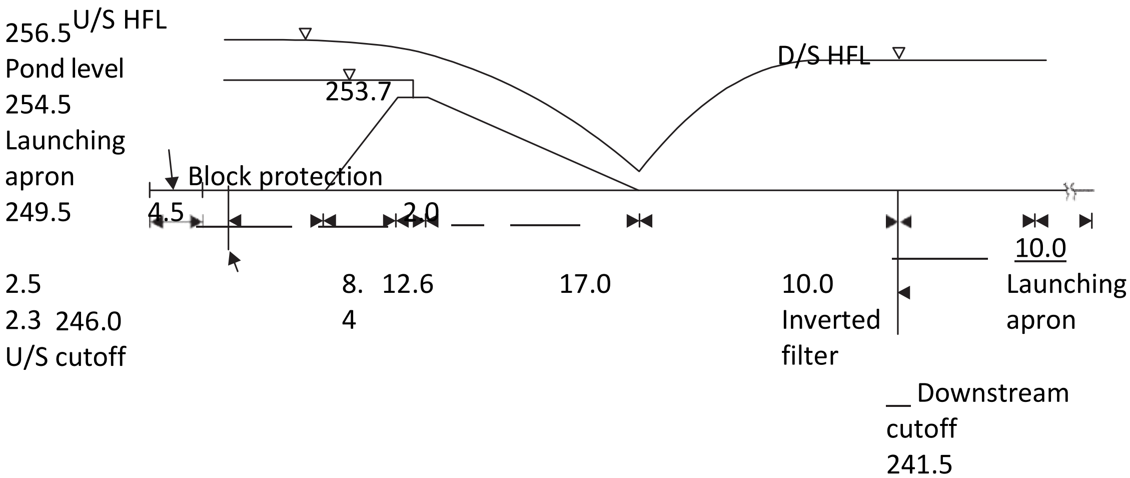


Fig. 13.7 Weir profile for Example 13.1

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 undersluice as shown in Fig. 13.1. 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 offtaking 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 condi- tions, 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 (1). 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 (1). 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.

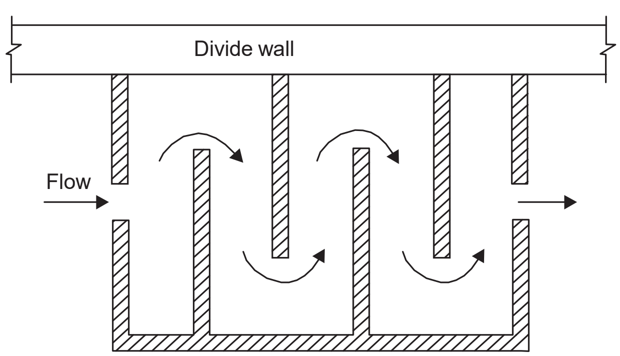
For the high stage of the river, the undersluices are discharging. At this condition, the water levels on the two sides are assumed to be different; the weir side level being higher by about 1.0 m.

It the river curvature is not favourable to sediment-free entry of water into the offtaking canal by inducing convex curvature opposite the head regulator, a second pocket of river sluice adjoining the undersluices improves flow conditions considerably. Such provision is useful in case of wide rivers to guide the river to flow centrally, minimising 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 (1).

FISH LADDER

Various kinds of fish are present in large rivers. Most of these fish migrate from the upstream to downstream in the beginning of the winter in search of warmth and return upstream before the monsoon for sediment-free water. While constructing a weir across a river, steps have to be taken to allow for the migration of fish. For this purpose, a narrow opening between the divide wall and the undersluices (where water is always present) is provided. Most fish can travel upstream, if the velocity of water does not exceed about 3.0 m/s. Hence, baffles or staggering devices are provided in the narrow opening adjacent to the divide wall. These openings are called fish ladder (or fishways or fish pass), (Fig. 13.8). It is advisable to know the requirements of the fish of the river, and the design of fish ladder should take intoaccount these requirements.

Weir portion



Undersluice portion

Fig. 13.8 Typical plan of a fish ladder

CANAL HEAD REGULATOR

A canal head regulator is required to serve the following functions:
To regulate the discharge into the offtaking canal, and
To control the entry of sediment into the canal.

The head regulator is usually aligned at an angle of 90° to 110° (Fig. 13.9) to the barrage axis. This orientation minimises entry of sediment into the offtaking canal and prevents backflow and stagnation zones in the undersluice pocket upstream of the regulator. The discharge through the regulator is controlled by gates. Steel gates of 6 to 8 m spans are generally used. However, larger spans can also be used in which case the gates are operated by electric winches.

The pond level in the undersluice pocket, upstream of the canal head regulator, is obtained by adding the working head of about 1.0 to 1.2 m to the designed full supply level of the canal. The level of the crest of the head regulator is obtained by subtracting from the pond level, the head over the crest required to pass the full supply discharge in the canal at the specific pond level. The crest of the regulator is always kept higher than the cill of the undersluices to prevent entry of sediment into the canal. If a sediment excluder is also provided in the undersluice pocket, the level of the crest of the head regulator should be decided keeping in view the design requirements of the sediment excluder in addition to the requirements of waterway, and the working head available.

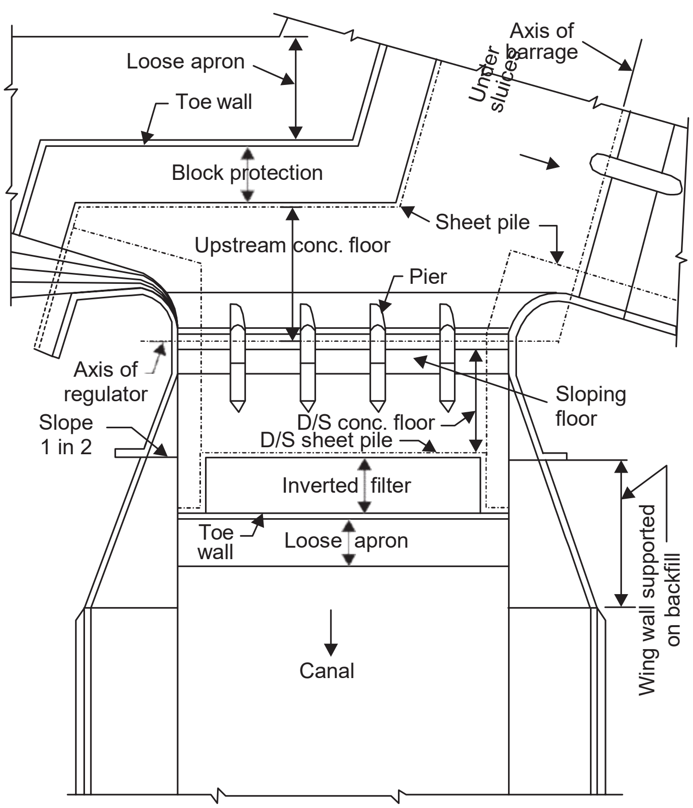


Fig. 13.9 Typical plan of a head regulator (6)

The width of waterway in the canal head regulator should be such that the canal can be fed its full supply with about 50 per cent of the working head provided (4). If the required width of waterway at the head regulator is more than the bed width of the canal, a converging transition is provided downstream of the regulator to attain the required canal width. The required head over the crest, H, for passing a discharge Q with an overall waterway L is worked out from Eq. (13.2).

The height of the gates is equal to the difference of the pond level and the crest level of the regulator. But during high floods, the water level in the river would be much higher than the pond level, and the flood water may spill over the gates. It would, obviously, be very uneconomical to provide gates up to the HFL. Besides the cost of heavier gates, the machinery required to operate them under large water pressures would also be expensive. To prevent such spilling of flood water into the canal, an RCC breast wall (Fig. 13.10) between the pond level and HFL, and spanning between adjacent piers is always provided. With this provision, the gate opening between the crest level and pond level is fully open when the gate is raised up fully, i.e., up to the pond level. The opening is fully closed when the gate is lowered to the crest of the regulator.

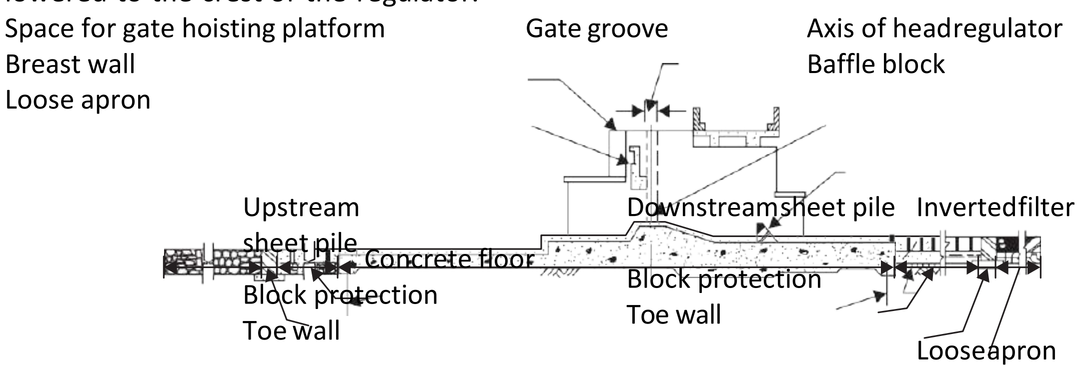


Fig. 13.10 Typical section of a head regulator (6)

Once the crest level, waterway, number of spans, and thickness of piers have been fixed, the head regulator is designed using the principles of weir design. The canal is generally kept closed when the highest flood passes through the river. This, obviously, would be the worst static condition and, hence, the floor thickness must be able to resist uplift pressures under this condition. The exit gradient for this condition should also be within safe limits. For economic reasons, the floor is designed such that it can support the uplift pressure by its weight and also the bending strength. For this purpose, the piers may have to be extended up to the floor to provide necessary support to the upward bending slab.

In the jump trough region, the worst condition of uplift may occur when some discharge is passing into the canal. The safety of this part of the floor should, therefore, be checked for different discharges including the maximum one also. The extension of the concrete floor upstream of the undersluices up to the end of the head regulator also reduces the uplift pressures on the downstream floor of the regulator.

A bridge and a working platform (for the operation of gates) are also constructed across the head regulator.

SEDIMENT CONTROL IN CANALS

Sediment entering into an offtaking canal, if excessive, causes silting, and thus, reduces canalcapacity. Even the fine suspended sediment, in power canals, would cause damage to the turbineblades and, therefore, needs removal prior to the canal water entering the power plant. As such, it is important to control amount of sediment entering into the offtaking canal. In all diversion structures, therefore, provision of adequate preventive or curative measures for sediment control is essential. If a canal offtakes from the outer side of a curved reach of a river, it draws much less sediment than the one offtaking from the inner side. This is due to the secondary flow which develops along the curved reach of a river.

Entry of sediment into the offtaking canal can be controlled by one of the following three methods of barrage regulation:

Still pond method,

Semi-open flow method, and

Wedge-flow method.

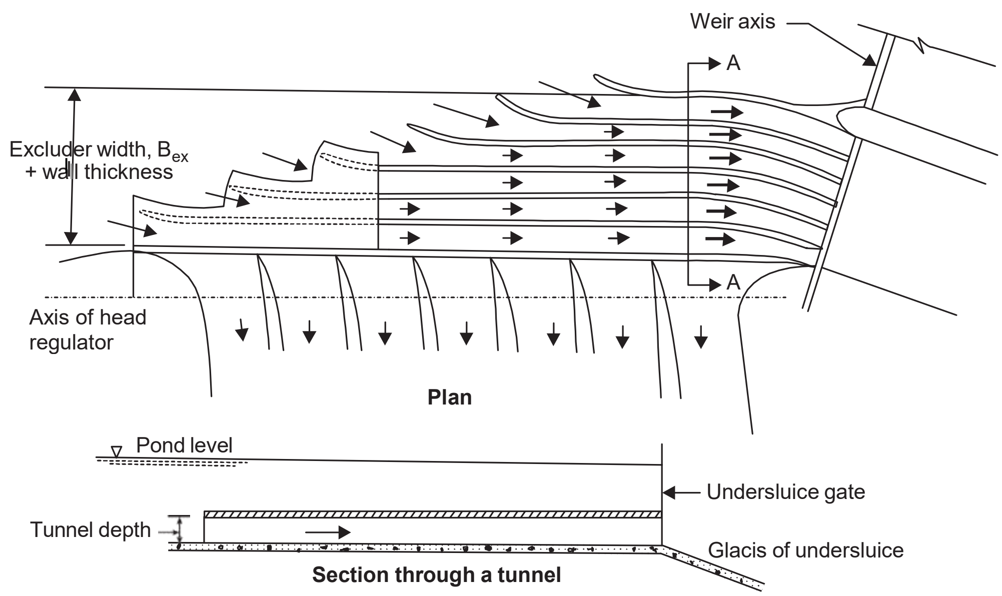
In the still pond method of the barrage regulation the undersluices are kept closed while the canal is taking its supplies, and the surplus water, if available, flows through some weir bays. This causes considerable reduction in the velocity of flow in the undersluice pocket which results in deposition of coarse sediment in the pocket and water containing much less (or no) sediment is drawn by the offtaking canal. However, with increasing amount of depositionof sediment in the pocket, the offtaking canal may start withdrawing sediment as well. At thisstage, the canal is closed and the deposited sediment is flushed downstream of the undersluices by opening the undersluices. This methods has been found satisfactory but requires closing ofcanal at some regular interval.

Alternatively, the undersluice gates are kept partially open while the canal is withdrawing its supplies. This semi-open flow method of barrage regulation results in continuous flushing of sediment through the undersluices while the canal is withdrawing top layer water which contains much less sediment. Besides requiring surplus water, this method results in two streams – one entering the canal and the other entering the undersluices which may generate enough turbulence in the pocket upstream of the head regulator and thus bring sediment into suspension. The suspended sediment may enter the canal. Also, the method would not work satisfactorily if there is no surplus water. This method is, therefore, not suitable except during floods.

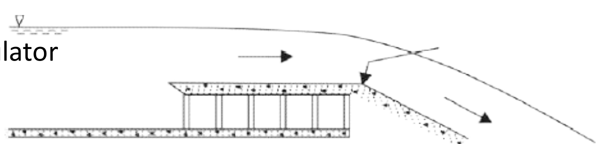
In the wedge-flow system of barrage regulation, the undersluices near the divide wall are opened more while those near the head regulator are opened less. This results in wedge-like flow cross-section which causes favourable curvature of flow in the undersluice pocket, and thus, reduces the amount of sediment entering the canal.

When the stream is carrying high sediment load, the sediment entry into the offtaking canal can be best checked by closure of the canal itself.

One of the most commonly used preventive measures is the sediment excluder (also known as silt excluder). The excluder is constructed in the river bed in front of the canal head regulator to prevent, as far as possible, excess sediment entering into the offtaking canal. Figure 13.11 shows a typical layout of a tunnel-type sediment excluder. The tunnels of the excluder, used for flushing the sediments, are parallel to the axis of the canal head regulator and are of different lengths, and the tunnels terminate at the end of the undersluice bays. Some kind of sediment control devices, such as skimming platform (6) and curved wings with sediment vanes (7) are provided in case of channels offtaking from main canals or branch canals for proportionate distribution of sediment. If the offtaking canal has already drawn more sediment, curative measures, such as construction of sediment ejectors, are adopted. Asediment ejector (or extractor), also known as silt ejector, is a curative measure and is constructed in the offtaking canal downstream of the canal head regulator to remove the excess sediment load which has entered the canal. Alternatively, a settling basin can be constructedin the offtaking canal for the purpose of sediment ejection.



Pond level
Crest of head regulator



Section A-A

Fig. 13.11 Typical layout of a sediment excluder

Sediment Excluders

The sediment concentration is maximum in the bottom layers of a stream. Tunnel-type sedimentexcluders (Fig. 13.11) prevent these bottom layers from entering the offtaking canal and allow only the top layers of the stream, containing relatively less sediment, to enter the offtaking canal. Such an excluder was first conceived by Elsden in 1922 and was first constructed by Nicholson for the Lower Chenab canal at Khanki headworks in Punjab. It was followed by another at Trimmu headworks. The tunnel-type sediment excluder, provided in front of the Nangal hydel channel, has been divided into two chambers – upper and lower – such that the heights of the upper and lower chambers are in the ratio of 2 : 1, and each of them has a separate control. The upper chamber is operated only during very high floods. This reduces the escape discharge at low flows.
The present design procedure for designing a sediment excluder is based on thumb rules evolved from past experiences on such structures. The minimum discharge passing through

the tunnels of the excluder is kept around 20 per cent of the canal discharge. The self-flushingvelocity in the tunnel ranges from 1.8 to 4 m/s depending upon the sediment size. Usually, 2 to 6 tunnels are provided in an excluder. The tunnels are to be accommodated in the space between the undersluice floor and the crest of the head regulator. The height of the tunnels is, thus, determined keeping in view convenience for inspection and repair, as well as self-flushing velocity. One can now estimate the width of waterway required for tunnels. This width is divided into a suitable number of tunnels such that a whole number of tunnels are accommodated in one undersluice bay. These tunnels are usually of rectangular cross- section and are bell-mouthed at the upstream end.

The water and sediment discharge carried by all the tunnels of an excluder should be the same. Also, the depth of all the tunnels are kept the same. Therefore, in accordance with the resistance and continuity equations, the width of the shorter tunnel will be smaller than that of the longer tunnel so that the head losses are the same in all tunnels.

The length of the tunnel nearest the head regulator must be equal to the length of the head regulator. Other tunnels are successively shorter in length such that the mouth of the one nearer the head regulator comes within the suction zone of the next tunnel so that no dead zone is left between the adjacent tunnels to cause sediment deposition. The excluder designed in this manner is usually tested through model study and is suitably modified, if required.

The design procedure outlined above takes into account the concepts of minimum energy loss and self-flushing velocity. It does not consider the actual sediment transport capacity of the tunnels for given set of conditions. It would, obviously, be more logical to compute the sediment load coming into the undersluice pocket and then design the excluder such that it is capable of carrying that sediment load without causing objectionable deposits of sediment. Garde and Pande (8) have suggested the following design procedure which takes into account the actual sediment transport capacity as well.

The velocity in the excluder tunnels, U_{ex} is chosen to lie between the critical velocity U_c (at which sediment starts moving) and the limit deposit velocity U_L (at which no sediment will be deposited) for the maximum sediment size, d_m. The critical velocity U_c is given as (9)

$$\frac{U_c}{\sqrt{\frac{\Delta\rho_s}{\rho} g d_m}} = 1.6\left(\frac{R_{ex}}{d_m}\right)^{1/8} \qquad (13.5)$$

and the limit deposit velocity U_L is given as (10)

$$U_L = F_1 \sqrt{8 g R_{ex} (S_s - 1)} \qquad (13.6)$$

where, $S_s = \frac{\rho_s}{\rho}$

ρ_s = the mass density of sediment,
ρ = the mass density of water,
R_{ex} = the hydraulic radius of the excluder tunnel.

and F_1 depends on the concentration and size distribution of sediment as shown in Fig. 13.12. Garde and Pande (8) suggested that for sediment size greater than 0.5 mm, F_1 varies between
and 1.0, and can be approximated as unity.

If the excluder velocity U_{ex} is greater than U_L there will be no deposition of sediment inthe excluder. However, the value of U_L is generally large and, hence, U_{ex} is usually less than U_L and, therefore, the excluder tunnels are partially blocked. The free flow area in such a casecan be calculated from

$$\frac{U_{fex}}{\sqrt{4gR_{fex}}} = \frac{U_L}{\sqrt{4gR_{ex}}} \qquad (13.7)$$

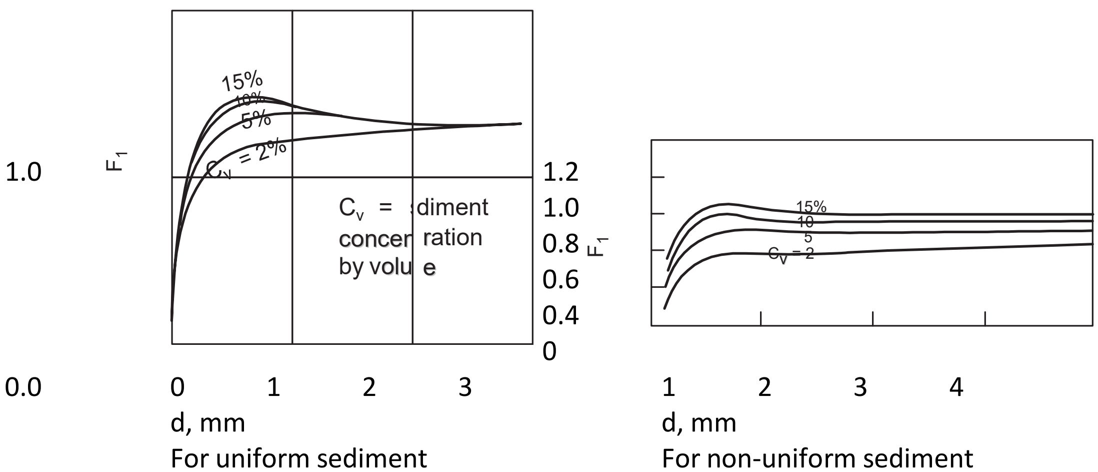


Fig. 13.12 Limit deposit velocity (10)

where, $$U_{fex} = \frac{Q_{ex}}{B_{ex}D_{fex}} \qquad (13.8)$$

and $$R_{fex} = \frac{B_{ex}D_{fex}}{2(B_{ex} + D_{fex})} \qquad (13.9)$$

Thus, combining Eqs. (13.6 – 13.9) one gets

$$\frac{Q_{ex}^2}{gD_{fex}^3 B_{ex}^2}\left(1 + \frac{2D_{fex}}{B_{ex}}\right) = 4F_1^2(S_s - 1) \qquad (13.10)$$

Here, the suffix ‘f’ refers to the free (i.e., unblocked) flow area available, and the suffix ‘ex’ refers to the excluder. B, D, and Q are, respectively, the width, depth, and the discharge of the excluder. Using the above equations, D_{fex} can be calculated. Hence, the blockage of the tunnels is obtained as $D_{ex} - D_{fex}$. One can also calculate U_{fex} from Eq. (13.8).

The bed load and suspended load going into the undersluice pocket are worked out by using suitable bed load relation and the velocity and sediment concentration profiles. Thus, the total sediment concentration (by weight) in the tunnels, C_{ex}, can be obtained. The sediment transport capacity of the tunnels C_t (i.e., sediment concentration by weight) must, obviously, be greater than C_{ex}. Further, C_t is obtained from (11).

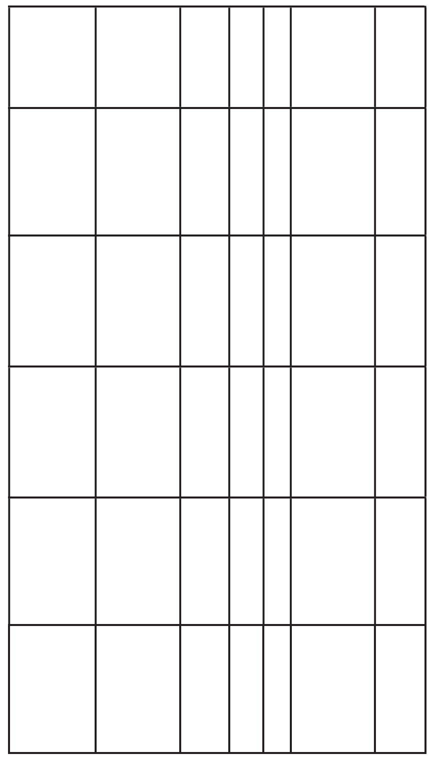
$$\frac{\text{l} \quad \text{I} \quad 0}{R_e \quad = M}^{s1} \qquad \sqrt{f} \quad P \quad C_t^{1/3} \tag{13.11}$$

$d/(4R_{fex})$

Here, R_e is the Reynolds number of flow [= U_{fex} $(4R_{fex})/\nu$], f the Darcy-Weisbach resistance coefficient, and I is a suitable function of d as given in Fig. 13.13. Further, the index s_1 is obtained from

$$s_1 = \frac{1}{0.89d^{1/3}} \quad (13.12)$$

in which, d (in mm) can be either equal to d_m or slightly less than d_m.



1000

100

10.0

I

1.0

0.1

0.01

0.1 0.2 0.4 0.6 0.81.0 2.0 3.0

d in mm

Fig. 13.13 Variation of I with d (11)

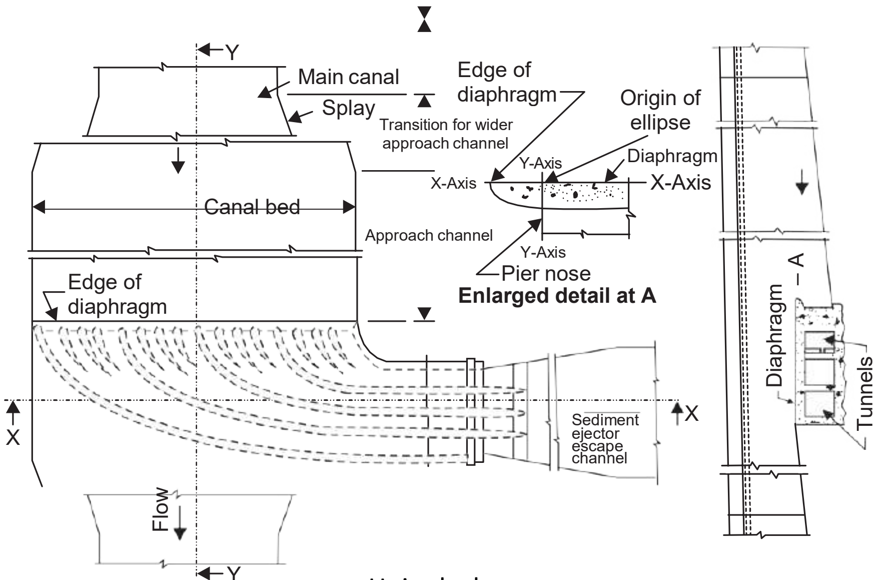
A suitable design for an excluder requires that the blockage be limited to a suitable value, say, about 35 per cent and also C_t is greater than C_{ex} in the excluder tunnels. Ideally, a design which satisfies all the conditions for minimum values of Q_{ex}, B_{ex}, and depth of tunnels will be the most suitable. However, there are some constraints on all these parameters. Therefore, one should choose the best combination satisfying all the requirements and constraints. Some margin of safety should always be provided for in order to account for the uncertainties in the sediment load computations. Following example illustrates the procedure for trial design of a sediment excluder based on the method given by Garde and Pande (8).

Sediment Ejector

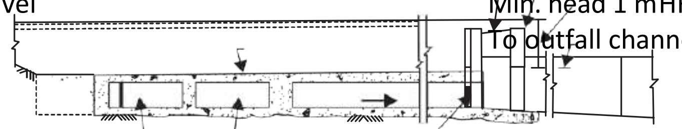
Sediment ejectors too take the advantage of the concentration distribution in a vertical by ejecting the near-bed water layers having the largest sediment concentration from the canal at a suitable location downstream of the head regulator. The approach channel upstream of the ejector should, preferably, be straight since a curved approach disturbs the uniform distribution of flow and sediment concentration across the channel in front of the ejector. The approach channel can be designed so that the suspended particles may move to lower layers. This will improve the efficiency of the ejector. The ejector should neither be too near nor too far from the head regulator. If the ejector is located too near the regulator, the residual turbulence may keep most of the sediment particles in suspension and, thus, prevent their ejection to the desired extent. If the ejector is sited too far downstream of the regulator, the sediment may get deposited between the regulator and the ejector and, thus, reduce the channel capacity. Besides, a longer reach (between the regulator and the ejector) will have to be wide enough to carry larger amount of discharge on account of the escape discharge required at the ejector.

A schematic diagram of the tunnel-type sediment ejector is shown in Fig. 13.15. The main components of an ejector include a diaphragm, tunnels, control structure, and an outfall channel. The diaphragm is so shaped that it causes least disturbance to the sediment distribution in the bottom layers of flow upstream of the ejector. Diaphragm level is fixed keeping in view the desired sediment size to be ejected, upstream and downstream bed levels of canal, size of tunnels, and the thickness of diaphragm. The lower side of the upstream end of the diaphragm is bell-mouthed. The canal bed is depressed below the ejector to facilitate further ejection of sediment. The ejector spans the entire width of the canal and is divided into a number of main tunnels which, in turn, are subdivided with turning vanes which gradually converge so as to accelerate the escaping flow. The width of ejector tunnels may be varied in order to keep the discharges in all the tunnels to be approximately the same. Generally, 10 to 20 per cent of the full supply discharge of the canal is adequate to remove the desired size and amount of sedimentas well as for flushing individual channels of the ejector. The tunnel dimensions at the entry and exit should be such that the resulting flow velocities would be adequate to carry the sediments of the desired size. In addition, the sub-tunnels should be contracted such that the exit velocities further increase by 10-15 per cent and should be in the range of 2.5 to 6 m/s depending on the size of the sediment to be ejected. The depth of tunnels should be kept about

1.8 to 2.2 m to facilitate inspection and repair. The ejector discharge is controlled by regulator gates. The outflow from the ejector is led to a natural drainage through an outfall channel which is designed to have a self-cleaning velocity. Sufficient drop between the full supply level of the outfall channel at its tail end and the normal high flood level of the natural stream is necessary for efficient functioning of the channel. The present design practice for sediment ejectors is also empirical with tunnel height being kept 20-25% of the depth of flow with an escape discharge of about 20 per cent of the full supply discharge of the canal downstream of the ejector. It is, therefore, essential that the proposed design be always model-tested before construction.



Plan at diaphragm level



Section X-X

Fig. 13.15 Typical layout of a sediment ejector

The efficiency (E) of the sediment ejector can be defined as (13)

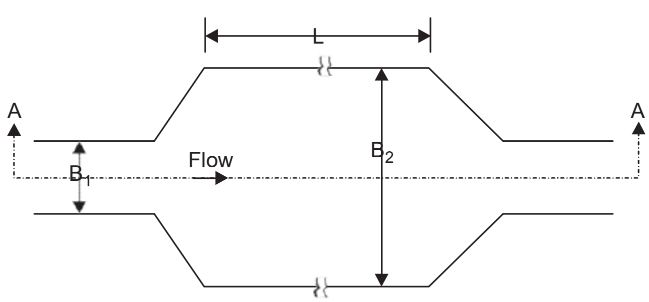
$$E = \frac{I_u - I_d}{I_u} \times 100 \text{ per cent} \quad (13.13)$$

Here, I_u and I_d refer, respectively, to the silt concentration in the canal at the upstream and the downstream of the ejector. A similar definition of efficiency can be used for sediment excluder too.

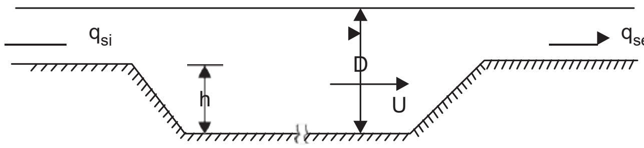
Recently, Vittal and Shivcharan Rao (14) suggested a rational method to decide on the height of the ejector diaphragm. The method is based primarily on the premises that : (i) the suspended load above the diaphragm only passes the ejector and enters the canal downstream of the ejector, and (ii) this suspended load (above the diaphragm) should be equal to the total sediment load transport capacity (i.e., the sum of the bed load and suspended load) of the canal downstream of the ejector, if it is to be neither silted nor scoured. In addition, the proposed method also assumed uniform size of sediment and validity of Rouse's equation, Eq. (7.34), for the variation of sediment concentration along a vertical and the logarithmic variation of velocity in sediment-laden flows. Also, river-bed material of coarser size is assumed to have been removedby the sediment excluder.

Settling Basin

In case of canals carrying fine sediment, the variation of sediment concentration will be almost uniform, i.e., the sediment will be moving more as suspended load than bed load. In such situations, the height of the ejector platform will have to be raised. This would result in much larger escape discharge that may not be desirable. An effective way of removing fine sediment from flowing water in a canal is by means of a settling basin in which flow velocity is reduced considerably by expanding the cross-sectional area of flow over the length of the settling basin, Fig. 13.17. The reduction in velocity, accompanied with reduction in bed shear and turbulence, stops the movement of the bed material, and also causes the suspended material to deposit on the bed of the basin. The material on the bed of the settling basin is suitably removed. The settling basin, a costly proposition, is suitable for power channels carrying fine sediment which may damage the turbine blades. The design of such a settling basin involves estimation of the dimensions of the settling basin and suitable method for removing the material from the bed of the settling basin. For known size (and, hence, fall velocity w) of the sediment particle and the depth of flow D, the time required for the particle on the water surface to settle on the bed of the basin would be (D/w) and if the particle moves with a horizontal velocity of flow U, the required length of the basin should be equal to or more than UD/w. To account for the reduction in fall velocity due to turbulence, the length of the basin so obtained may be increased by about 20%.



Plan



Section AA

Fig. 13.17 Settling basin - definition sketch

The efficiency of removal of sediment η by a settling basin is defined as

$$\eta = \frac{q_{si} - q_{se}}{q_{si}} = 1 - \frac{q_{se}}{q_{si}} \tag{13.26}$$

Here, q_{si} and q_{se} are, respectively, the amount of sediment of a given size entering and leaving the settling basin in a unit time. Dobbins (15) obtained an analytical solution for the estimation of η by assuming : (i) the longitudinal concentration gradient zone, (ii) uniform velocity distribution for flow in the basin, and (iii) invariant diffusion coefficient over the flow section in the basin.

These methods can be used suitably for estimating the efficiency of sediment removal, η for sediment size d in a given settling basin. However, for designing a suitable settling basin to achieve the desired efficiency of sediment removal, η for sediment size d and given flow rate Q, one needs to select suitable combinations of L, B_2, and D for the settling basin and obtain the values of η for each of these combinations. The one which gives η higher than the desired value of η can be selected. At times, these different methods would give very different results and one should use the results judiciously.

The performance of a settling basin is adversely affected if the flow conditions in the basin are relatively turbulent or there is some amount of 'short-circuiting' of flow on account of separation due to expansion in cross-section of flow. Therefore, a suitable expanding transition with two (or more) splitter plates at the entrance of the basin and a contracting transition at the outlet end of the basin are always provided.

Further, a suitable provision to remove the deposited material from the bed of the basin is to be made. For this, one can have two settling basins parallel to each other so that while the material from one basin is removed, the other is in operation. Alternatively, the bed of the basin may be divided into suitable number of hopper-shaped chambers and a suitable pipe outlet in each of these chambers is provided. The deposited sediment may be flushed away through these outlets.

RIVER TRAINING FOR CANAL HEADWORKS

River training structures for canal headworks are required for the following purposes (13):
To prevent outflanking of the structure,
To minimise possible cross-flow through the barrage or weir which may endanger the structure and protection works.
To prevent flooding of the riverine lands upstream of the barrages and weirs, and
To provide favourable curvature of flow at the head regulator from the consideration of entry of sediment into the canal.
The following types of river training structures are usually provided for weirs (13):
Guide banks,
Approach embankments,
Afflux embankments, and
Groynes or spurs.
The purpose of guide banks is to narrow down and restrict the course of a river so that the river flows centrally through the weir constructed across it without damaging the structure and its approaches. The alignment of guide banks should be determined such that the pattern of flow at the head regulator induces favourable curvature of flow minimising sediment entry into the canal system (19). More details of guide banks have been included in Chapter 12 and a typical sketch of river training structures provided at weirs and salient details of guide banks are given in Figs. 13.21 and 13.22. While constructing weirs on alluvial rivers, the natural waterway is restricted from economic and flow considerations, and the unbridged width of river is blocked by means of approach embankments. Depending upon the distance between the guide bank and the extent of the alluvial belt, the river may form either one or two meander loops (Fig. 13.21). The approach embankments, aligned with the weir axis, should extend up to a point beyond the range of the worst anticipated loop (19).
Afflux embankments are earthen embankments which extend from both the abutments (or the approach embankments) and are connected on the upstream to the ground above the affluxed highest flood level (or to flood embankments, if existing).

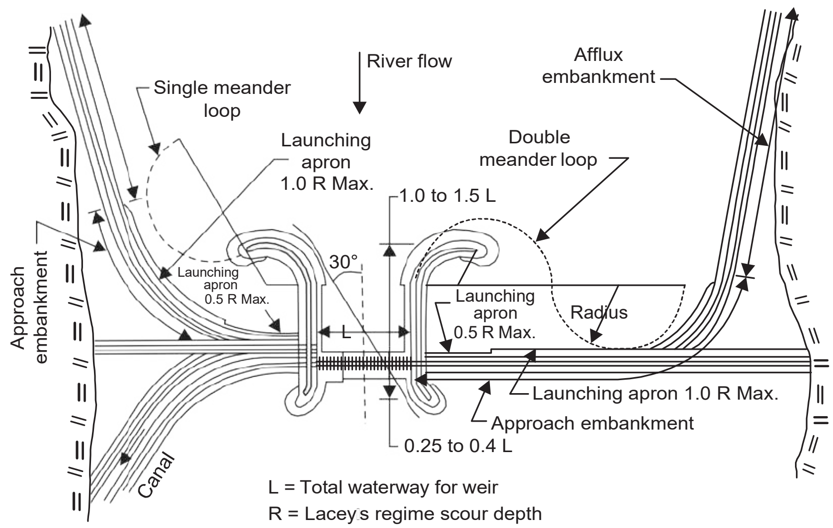
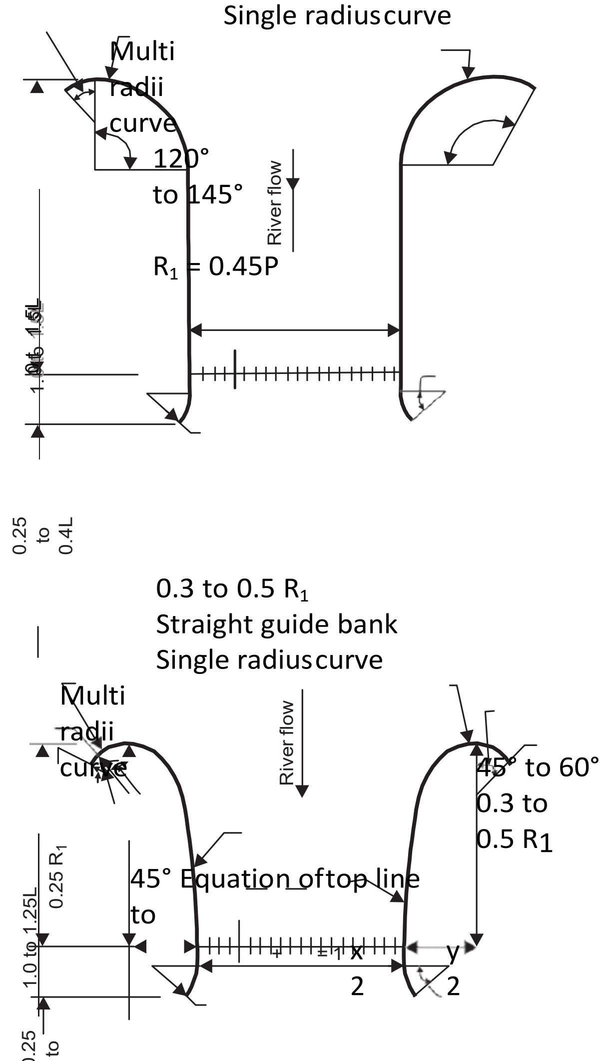


Fig. 13.21 Typical layout of river training structures for canal headworks



a b a

b

L

0.4L

b Origin

0.3 to 0.5 R_1

45° to 60°

Elliptical guide bank

Fig. 13.22 Geometrical shape of guide banks

A top width of about 6 to 9 m and freeboard of 1.0 to 1.5 m above the highest flood level (for a 1-in-500 years flood) are usually provided for these river training works (19). Besides, stone pitching, launching aprons, etc. are also provided in the usual manner as described in Chapter 12.

3.1.2Design Consideration of canals, canal structures and cross drainage Works

CANALIRRIGATION

CANALS

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 canals.

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. Canals can be classified in many ways.

Based on the nature of source of supply, a canal can be either a permanent or an inundation canal. A permanent canal has a continuous source of water supply. Such canals are also called perennial canals. An inundation canal draws its supplies from a river only during the high stages of the river. Such canals do not have any headworks for diversion of river water to the canal, but are provided with a canal head regulator.

Depending on their function, canals can also be classified as: (i) irrigation, (ii) navigation, (iii) power, and (iv) feeder canals. An irrigation canal carries water from its source to agriculturalfields. Canals used for transport of goods are known as navigation canals. Power canals are used to carry water for generation of hydroelectricity. A feeder canal feeds two or more canals.

A canal can serve more than one function. 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.

Similarly, irrigation canals can also be utilized for the transportation of goods and serve as navigation canals. Inland navigation forms a cheap means of transportation of goods and, hence, must be developed. However, in India, inland navigation has developed only to a limited extent. This is mainly due to the fact that irrigation canals generally take their supplies from alluvial rivers and, as such, must flow with sufficient velocity to prevent siltation of the canal. Such velocities make upstream navigation very difficult. Besides, the canals are generally aligned on the watershed so that water may reach the fields on both sides by flow. This alignment may not be suitable for navigation which requires the canal to pass through the areas in the vicinity of industries.

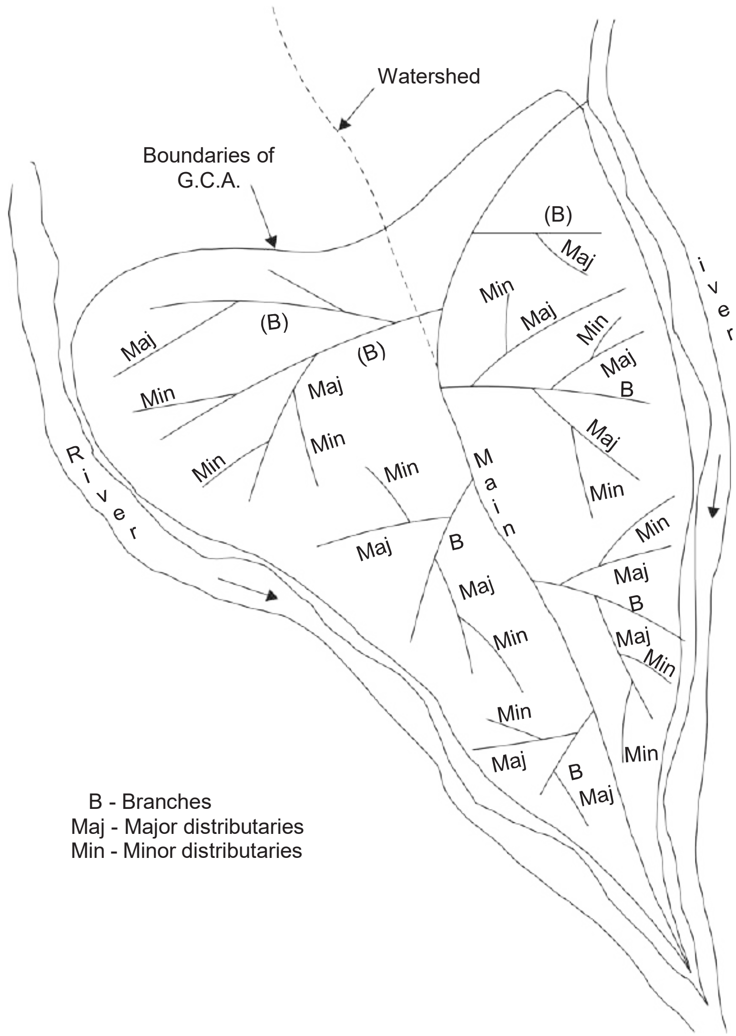


Fig. 5.1 Layout of an irrigation canal network

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.

Branch canals (also called 'branches') take their supplies from the main canal. Branch canals generally carry a discharge higher than 5 m^3/s and act 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.

Major distributaries (also called 'distributaries' or rajbaha) carry 0.25 to 5 m^3/s of discharge. These distributaries take their supplies generally from the branch canal and some times from the main canal. The distributaries feed either watercourses through outletsor minor distributaries.
Minor distributaries (also called 'minors') are small canals which carry a discharge lessthan 0.25 m^3/s and feed the watercourses for irrigation. They generally take their supplies from major distributaries or branch canals and rarely from the main canals.
A watercourse 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 areato be irrigated through the outlet.

COMMAND AREAS

Gross command area (or GCA) is the total area which can be economically irrigated from an irrigation system without considering the limitation on the quantity of available water. It includes the area which is, otherwise, uncultivable. For example, ponds and residential areas are uncultivable areas of gross command area. An irrigation canal system lies in a doab (i.e., the area between two drainages), and can economically irrigate the doab. It is, obviously, uneconomical to use the irrigation system to irrigate across the two drainages. Thus, the boundaries of the gross command of an irrigation canal system are fixed by the drainages on either side of the irrigation canal system.

The area of the cultivable land in the gross command of an irrigation system is called cultivable command area (CCA) and includes all land of the gross command on which cultivation is possible. At any given time, however, all the cultivable land may not be actually under cultivation. Therefore, sometimes the CCA is divided into two categories: cultivated CCA andcultivable but not cultivated CCA.
Intensity of irrigation is defined as the percentage of CCA which is proposed to be annually irrigated. Till recently, no irrigation system was designed to irrigate all of its cultivable command every year. This practice reduces the harmful effects of over-irrigation such as wate r logging and malaria. Also, due to the limitations on the quantity of available water, it is preferred to provide protection against famine in large areas rather than to provide intensive irrigation of a smaller area. The intensity of irrigation varied between 40 per cent to 60 percent till recently. This needs to be raised to the range of 100 per cent to 180 per cent by cultivatingparts of CCA for more than one crop in a year and through improved management of the existing system. Future projects should be planned for annual intensities of 100 percent to 180 per cent depending on the availability of total water resources and land characteristics.
The cultivable command area multiplied by the intensity of irrigation (in fraction) gives the actual area to be irrigated. The water requirements of the controlling crops of two crop seasons may be quite different. As such, the area to be irrigated should be calculated for each crop season separately to determine the water requirements.

PLANNING OF AN IRRIGATION CANAL SYSTEM

Planning of an irrigation canal project includes the determination of: (i) canal alignment, and
(ii) the water demand. The first step in the planning of an irrigation canal project is to carry out a preliminary survey to establish the feasibility or otherwise of a proposal. Once the feasibility of the proposal has been established, a detailed survey of the area is carried out and, thereafter, the alignment of the canal is fixed. The water demand of the canal is, then, workedout.

Preliminary Survey

To determine the feasibility of a proposal of extending canal irrigation to a new area, information on all such factors which influence irrigation development is collected during the preliminary (or reconnaissance) survey. During this survey all these factors are observed or enquired from the local people. Whenever necessary, some quick measurements are also made.

The information on the following features of the area are to be collected:

Type of soil,

Topography of the area,

Crops of the area,

Rainfall in the area,

Water table elevations in the area,

Existing irrigation facilities, and

General outlook of the cultivators with respect to cultivation and irrigation.

The type of soil is judged by visual observations and by making enquiries from the local people. The influence of the soil properties on the fertility and water holding capacity has already been disscused.

For a good layout of the canal system, the command area should be free from too many undulations. This requirement arises from the fact that a canal system is essentially a gravity flow system. However, the land must have sufficient longitudinal and cross slopes for the channels to be silt-free. During the preliminary survey, the topography of the area is judged by visual inspection only.

Water demand after the completion of an irrigation project would depend upon the crops being grown in the area. The cropping pattern would certainly change due to the introduction of irrigation, and the possible cropping patterns should be discussed with the farmers of the area.

The existing records of rain gauge stations of the area would enable the estimation of the normal rainfall in the area as well as the probability of less than normal rainfall in the area. This information is, obviously, useful in determining the desirability of an irrigation project in the area.

Water table elevation can be determined by measuring the depth of water surface in a well from the ground with the help of a measuring tape. Water table elevation fluctuates considerably and information on this should be collected from the residents of the area and checked by measurements. Higher water table elevations in an area generally indicate good rainfall in the area as well as good soil moisture condition. Under such conditions, the demand for irrigation would be less and introduction of canal irrigation may cause the water table torise up to the root zone of the crops. The land is then said to be waterlogged and the productivity of such land reduces considerably. Waterlogged land increases the incidence of malaria in the affected area. Thus, areas with higher water table elevation are not suitable for canal irrigation.

Because of limited financial and hydrological resources, an irrigation project should be considered only for such areas where maximum need arises. Areas with an extensive network of ponds and well systems for irrigation should be given low priority for the introduction of canal irrigation.

The success or failure of an otherwise good irrigation system would depend upon the attitudes of the farmers of the area. Enlightened and hard-working cultivators would quickly adapt themselves to irrigated cultivation to derive maximum benefits by making use of improved varieties of seeds and cultivation practices. On the other hand, conservative farmers will have to be educated so that they can appreciate and adopt new irrigated cultivation practices.

The information collected during preliminary survey should be carefully examined to determine the feasibility or otherwise of introducing canal irrigation system in the area. If the result of the preliminary survey is favourable, more detailed surveys would be carried out and additional data collected.

Detailed Survey

The preparation of plans for a large canal project is simplified in a developed area because of the availability of settlement maps (also called shajra maps having scale of 16 inches to a mile i.e., 1/3960 to 1/4000) and revenue records in respect of each of the villages of the area. The settlement maps show the boundaries and assigned numbers of all the fields of the area, location of residential areas, culturable and barren land, wells, ponds, and other features of the area. Usually for every village there is one settlement (or shajra) map which is prepared on a piece of cloth. These maps and the revenue records together give information on total land area, cultivated area, crop-wise cultivated area and the area irrigated by the existing ponds andwells.

With the help of settlement maps of all the villages in a doab, a drawing indicating distinguishing features, such as courses of well-defined drainages of the area, is prepared. On this drawing are then marked the contours and other topographical details not available on the settlement maps but required for the planning of a canal irrigation project. Contours aremarked after carrying out 'levelling' survey of the area.

The details obtained from the settlement maps should also be updated in respect of developments such as new roads, additional cultivated area due to dried-up ponds, and so on. In an undeveloped (or unsettled) area, however, the settlement maps may not be available and the plans for the canal irrigation project will be prepared by carrying out engineering survey of the area.

One of the most important details from the point of view of canal irrigation is the watershed which must be marked on the above drawing. Watershed is the dividing line between the catchment areas of two drains and is obtained by joining the points of highest elevation on successive cross-sections taken between any two streams or drains. Just as there would be the main watershed between two major streams of an area, there would be subsidiary watersheds between any tributary and the main stream or between any two adjacent tributaries.

ALIGNMENT OF IRRIGATION CANALS

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 command of such irrigation channels would be the drainages on either side of the channel. Aligning a canal (main, branch as well as distributaries) 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. This can be illustrated by Fig.

5.2 in which the main canal takes off from a river at P and mounts the watershed at Q. Let the canal bed level at P be 400 m and the elevation of the highest point N along the section MNP be 410 m. Assuming that the ground slope is 1 m per km, the distance of the point Q (395 m) on the watershed from N would be 15 km. If the required canal bed slope is 25 cm per km, the length PQ of the canal would be 20 km. Between P and Q, the canal would cross small streams and, hence, construction of cross-drainage structures would be necessary for this length. In fact, the alignment PQ is influenced considerably by the need of providing suitable locations for the cross-drainage structures. The exact location of Q would be determined by trial so that the alignment PQ results in an economic as well as efficient system. Further, on the watershed side of the canal PQ, the ground is higher than the ground on the valley side (i.e., the river side). Therefore, this part of the canal can irrigate only on one side (i.e., the river side) of the canal.

Watershed

P (400)

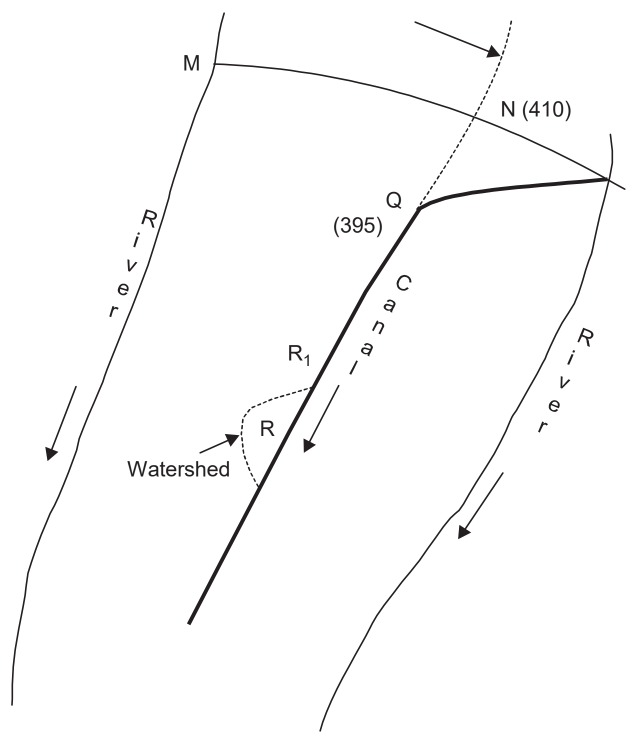
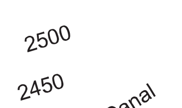


Fig. 5.2 Head reach of a main canal in plains

Once a canal has reached the watershed, it is generally kept on the watershed, except in certain situations, such as the looping watershed at R in Fig. 5.2. In an effort to keep the canal alignment straight, the canal may have to leave the watershed near R. The area between the canal and the watershed in the region R can be irrigated by a distributary which takes off at R_1 and follows the watershed. Also, in the region R, the canal may cross some small streams and, hence, some cross-drainage structures may have to be constructed. If watershed is passing through villages or towns, the canal may have to leave the watershed for some distance.

In hilly areas, the conditions are vastly different compared to those of plains. Rivers flow in valleys well below the watershed or ridge, and it may not be economically feasible to take the channel on the watershed. In such situations, contour channels (Fig. 5.3) are constructed. Contour channels follow a contour while maintaining the required longitudinal slope. It continues like this and as river slopes are much steeper than the required canal bed slope the canal encompasses more and more area between itself and the river. It should be noted that the more fertile areas in the hills are located at lower levels only.



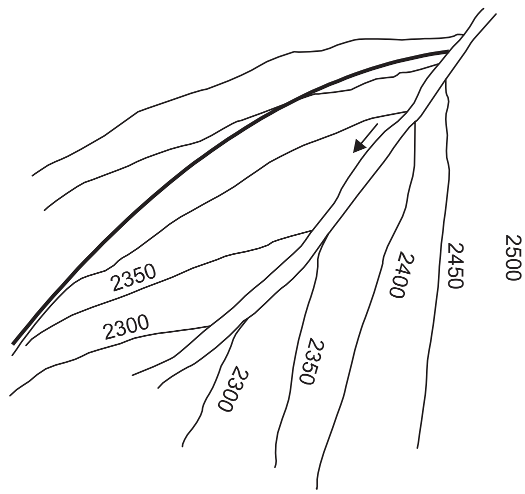


Fig. 5.3 Alignment of main canal in hills

In order to finalize the channel network for a canal irrigation project, trial alignments of channels are marked on the map prepared during the detailed survey. A large-scale map isrequired to work out the details of individual channels. However, a small-scale map depicting the entire command of the irrigation project is also desirable. The alignments marked on the map are transferred on the field and adjusted wherever necessary. These adjustments are transferred on the map as well. The alignment on the field is marked by small masonry pillarsat every 200 metres. The centre line on top of these pillars coincides with the exact alignment. In between the adjacent pillars, a small trench, excavated in the ground, marks the alignment.

CURVES IN CANALS

Because of economic and other considerations, the canal alignment does not remain straight all through the length of the canal, and curves or bends have to be provided. The curves cause disturbed flow conditions resulting in eddies or cross currents which increase the losses. In a curved channel portion, the water surface is not level in the transverse direction. There is a slight drop in the water surface at the inner edge of the curve and a slight rise at the outer edge of the curve. This results in slight increase in the velocity at the inner edge and slight decrease in the velocity at the outer edge. As a result of this, the low-velocity fluid particles near the bed move to the inner bank and the high-velocity fluid particles near the surface gradually cross to the outer bank. The cross currents tend to cause erosion along the outer bank. The changes in the velocity on account of cross currents depend on the approach flow condition and the characteristics of the curve. When separate curves follow in close succession, either in the same direction or in the reversed direction, the velocity changes become stillmore complicated.

Therefore, wherever possible, curves in channels excavated through loose soil should be avoided. If it is unavoidable, the curves should have a long radius of curvature. The permissible minimum radius of curvature for a channel curve depends on the type of channel, dimensions of cross-section, velocities during full-capacity operations, earth formation along channel alignment and dangers of erosion along the paths of curved channel. In general, the permissible minimum radius of curvature is shorter for flumes or lined canals than earth canals, shorter for small cross-sections than for large cross-sections, shorter for low velocities than for high velocities, and shorter for tight soils than for loose soils. Table 5.1 indicates the values of minimum radii of channel curves for different channel capacities.

Table 5.1 Radius of curvature for channel curves (1)

Channel capacity (m^3/s)	Minimum radius of curvature (metres)
Less than 0.3	100
0.3 to 3.0	150
0.3 to 15.0	300
15.0 to 30.0	600
30.0 to 85.0	900
More than 85	1500

DUTY OF WATER

For proper planning of a canal system, the designer has to first decide the 'duty of water' in the locality under consideration. Duty is defined as the area irrigated by a unit discharge of water flowing continuously for the duration of the base period of a crop. The base period of a crop is the time duration between the first watering at the time of sowing and the last watering before harvesting the crop. Obviously, the base period of a crop is smaller than the crop period. Duty is measured in hectares/m^3/s. The duty of a canal depends on the crop, type of soil, irrigation and cultivation methods, climatic factors, and the channel conditions.

By comparing the duty of a system with that of another system or by comparing it with the corresponding figures of the past on the same system, one can have an idea about the performance of the system. Larger areas can be irrigated if the duty of the irrigation system is improved. Duty can be improved by the following measures:

The channel should not be in sandy soil and be as near the area to be irrigated as possible so that the seepage losses are minimize. Wherever justified, the channel may be lined.

The channel should run with full supply discharge as per the scheduled program so that farmers can draw the required amount of water in shorter duration and avoid the tendency of unnecessary over irrigation.

Proper maintenance of watercourses and outlet pipes will also help reduce losses, and thereby improve the duty.

Volumetric assessment of water makes the farmer to use water economically. This is, however, more feasible in well irrigation.

Well irrigation has higher duty than canal irrigation due to the fact that water is used economically according to the needs. Open wells do not supply a fixed discharge and, hence, the average area irrigated from an open well is termed its duty.

Between the head of the main canal and the outlet in the distributary, there are losses due to evaporation and percolation. As such, duty is different at different points of the canal system. The duty at the head of a canal system is less than that at an outlet or in the tail end region of the canal. Duty is usually calculated for the head discharge of the canal. Duty calculated on the basis of outlet discharge is called 'outlet discharge factor' or simply 'outlet factor' which excludes all losses in the canal system.

CANAL LOSSES

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 headwork's 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 headwork's depending upon the type of soil through which canal runsand 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/s per million square metres of either wetted perimeter or the exposed water surface area. Conveyance loss can be calculated using the values given in Table 5.2. Generally, the total loss (due to seepage and evaporation) per million square metres of water surface varies from 2.5 m^3/s for ordinary clay loam to 5.0 m^3/s for sandy loam. The following empirical relation has also been found to give comparable results (2).

$q_l = (1/200)\,(B + h)^{2/3}$ (5.1)

Table 5.2 Conveyance losses in canals (1)

Material	Loss in m^3/s per million square metres of wetted perimeter (or watersurface)
Impervious clay loam	0.88 to 1.24
Medium clay loam underlaid with hard pan at depth of not over 0.60 to 0.90 m below bed	1.24 to 1.76
Ordinary clay loam, silty soil or lava ash loam	1.76 to 2.65
Gravelly or sandy clay loam, cemented gravel, sand and clay Sandy loam Loose sand Gravel sand Porous gravel soilGravels	2.65 to 3.53
	3.53 to 5.29
	5.29 to 6.17
	7.06 to 8.82 8.82 to 10.58
	10.58 to 21.17

In this relation, q_l is the loss expressed in m^3/s per kilometer length of canal and B and h are, respectively, canal bed width and depth of flow in metres.

ESTIMATION OF DESIGN DISCHARGE OF A CANAL

The amount of water needed for the growth of a crop during its entire crop-growing period is known as the water requirement of the crop, and is measured in terms of depth of water spread over the irrigated area. This requirement varies at different stages of the growth of the plant. The peak requirement must be obtained for the period of the keenest demand. One of the methods to decide the water requirement is on the basis of kor watering.

When the plant is only a few centimeters high, it must be given its first watering, called the kor watering, in a limited period of time which is known as the kor period. If the plants do not receive water during the kor period, their growth is retarded and the crop yield reduces considerably. The kor watering depth and the kor period vary depending upon the crop and the climatic factors of the region. In UP, the kor watering depth for wheat is 13.5 cm and the kor period varies from 8 weeks in north-east UP (a relatively dry region) to 3 weeks in the hilly region (which is relatively humid). For rice, the kor watering depth is 19 cm and the kor period varies from 2 to 3 weeks.

If D represents the duty (measured in hectares/m^3/s) then, by definition,

1 m^3/s of water flowing for b (i.e., base period in days) days irrigates D hectares.

1 m^3/s of water flowing for 1 day (i.e., 86400 m^3 of water) irrigates D/b hectares This volume (i.e., 86400 m^3) of water spread over D/b hectares gives the water depth.

$$= \frac{86400}{(D / b) \times 10^4} = 8.64\ b/D \text{ (metres)} \qquad (5.2)$$

For the purpose of designing on the basis of the keenest demand (i.e., the kor period requirement) the base period b and the water depth are replaced by the kor period and kor water depth, respectively.

The kor period for a given crop in a region depends on the duration during which there is likelihood of the rainfall being smaller than the corresponding water requirement. Accordingly, the kor period is least in humid regions and more in dryer regions. The kor depth requirement must be met within the kor period. As such, the channel capacity designed on the basis of kor period would be large in humid regions and small in dry regions. Obviously, this method of determining the channel capacity is, therefore, not rational, and is not used in practice.

A more rational method to determine the channel capacity would be to compare evapotranspiration and corresponding effective rainfall for, say, 10-day (or 15-day) periods of the entire year and determine the water requirement for each of these periods. The channel capacity can then be determined on the basis of the peak water requirement of the 10-day (or 15-day) periods. This method has already been explained in Sec. 3.8.

CANAL OUTLETS

When the canal water has reached near the fields to be irrigated, it has to be transferred to the watercourses. At the junction of the watercourse and the distributary, an outlet is provided. An outlet is a masonry structure through which water is admitted from the distributary into a watercourse. It also acts as a discharge measuring device. The discharge though an outlet is usually less than 0.085 m^3/s (3). It plays a vital role in the warabandi system (see Sec. 5.11) of distributing water. Thus, an outlet is like a head regulator for the watercourse.

The main objective of providing an outlet is to provide ample supply of water to the fields, whenever needed. If the total available supply is insufficient, the outlets must be such that equitable distribution can be ensured. The efficiency of an irrigation system depends on the proper functioning of canal outlets which should satisfy the following requirements (3):

The outlets must be strong and simple with no moving parts which would require periodic attention and maintenance.

The outlets should be tamper-proof and if there is any interference in the function- ing of the outlet, it should be easily detectable.

The cost of outlets should be less as a large number of these have to be installed in an irrigation network.

The outlet should be able to draw sediment in proportion to the amount of water withdrawn so that there is no silting or scouring problem in the distributary down-stream of the outlet.

The outlets should be able to function efficiently even at low heads.

The choice of type of an outlet and its design are governed by factors such as water distribution policy, water distribution method, method of water assessment, sources of supply, and the working of the distributary channel.

Water may be distributed on the basis of either the actual area irrigated in the previous year or the actual culturable command area. The discharge from the outlet should be capable of being varied in the first case, but, can remain fixed in the second. The method of water distribution may be such that each cultivator successively receives water for a duration in proportion to his area. Or, alternatively, all the cultivators share the outlet discharge simultaneously. The first system is better as it results in less loss of water. The outlet capacityis decided keeping in view the method of water distribution.

If the assessment is by volume, the outlet discharge should remain constant and not change with variation in the water levels of the distributary and the watercourse. On the other hand, if water charges are decided on the basis of area, the variation in the outlet capacity with water levels of the distributary and watercourse is relatively immaterial.

With a reservoir as supply source, the cultivators can be provided water whenever needed and, hence, the outlets should be capable of being opened or closed. The outlets generally remain open if the supply source is a canal without storage so that water is diverted to the field when the canal is running.

At times, the amount of water in the main canal may not be sufficient to feed all the channels simultaneously to their full capacity. As such, either all the channels may run with low discharge or groups of channels may be supplied their full capacity by rotation. In the first case, the outlets must be able to take their proportionate share even with large variations in the discharge of the distributary channel. In the second case, the outlets must be such that the required amount of water is available for all the channels being fed with their full capacity.

It should be noted that whereas the cultivator prefers to have outlets capable of supplying constant discharge, the canal management would prefer that the outlets supply variable discharge depending upon the discharge in the distributary channel so that the tail end of the channel is neither flooded nor dried. Obviously, both these requirements cannot be fulfilled simultaneously.

Types of Outlet

Canal outlets are of the following three types:

Non-modular outlets,

Semi-modular outlets, and

Modular outlets.

Non-modular outlets are those whose discharge capacity depends on the difference of water levels in the distributary and the watercourse. The discharge through non-modular outlets fluctuates over a wide range with variations in the water levels of either the distributary or the watercourse. Such an outlet is controlled by a shutter at its upstream end. The loss of head in a non-modular outlet is less than that in a modular outlet. Hence, non-modular outlets are very suitable for low head conditions. However, in these outlets, the discharge may vary even when the water level in the distributary remains constant. Hence, it is very difficult to ensure equitable distribution of water at all outlets at times of keen demand of water.

The discharge through a semi-modular outlet (or semi-module or flexible outlet) depends only on the water level in the distributary and is unaffected by the water level in the watercourse provided that a minimum working head required for its working is available. A semi-module is more suitable for achieving equitable distribution of water at all outlets of a distributary. The only disadvantage of a semi-modular outlet is that it involves comparatively greater loss of head.

Modular outlets are those whose discharge is independent of the water levels in the distributary and watercourse, within reasonable working limits. These outlets may or may not have moving parts. In the latter case, these are called rigid modules. Modular outlets with moving parts are not simple to design and construct and are, hence, expensive.

A modular outlet supplies fixed discharge and, therefore, enables the farmer to plan his irrigation accordingly. However, in case of excess or deficient supplies in the distributary, the tail-end reach of the distributary may either get flooded or be deprived of water. This is due to the reason that the modular outlet would not adjust its discharge corresponding to the water level in the distributary. But, if an outlet is to be provided in a branch canal which is likely to run with large fluctuations in discharge, a modular outlet would be an ideal choice. The outlet would be set at a level low enough to permit it to draw its due share when the branch is running with low supplies. When the branch has to carry excess supplies to meet the demands of the distributaries, the discharge through the modular outlet would not be affected and the excess supplies would reach up to the desired distributaries. Similarly, if an outlet is desired to be located upstream of a regulator or a raised crest fall, a modular outlet would be a suitable choice.

Parameters for Studying the Behaviour of Outlets

Flexibility

The ratio of the rate of change of discharge of an outlet (dQ_0/Q_0) to the rate of change of discharge of the distributary channel (dQ/Q) (on account of change in water level) is termed the flexibility which is designated as F. Thus,

$F = (dQ_0/Q_0)/(dQ/Q)$ (5.3)

Here, Q and Q_0 are the flow rates in the distributary channel and the watercourse, respectively. Expressing discharge Q in the distributary channel in terms of depth of flow h in the channel as

$$Q = C_1 h^n$$

one can obtain

$$\frac{dQ}{Q} = n\frac{dh}{h}$$

Similarly, the discharge Q_0 through the outlet can be expressed in terms of the head H on the outlet as

$$Q_0 = C_2 H^m$$

which gives

$$\frac{dQ_0}{Q_0} = m\frac{dH}{H}$$

Here, m and n are suitable indices and C_1 and C_2 are constants. Thus,

$$F = \frac{m}{n}\,\frac{h}{H}\,\frac{dH}{dh} \qquad (5.4)$$

For semi-modular outlets, the change in the head dH at an outlet would be equal to the change in the depth of flow dh in the distributary. Therefore,

$$F = \frac{m}{n}\,\frac{h}{H} \qquad (5.5)$$

If the value of F is unity, the rate of change of outlet discharge equals that of the distributary discharge. For a modular outlet, the flexibility is equal to zero. Depending upon the value of F, the outlets can be classified as: (i) proportional outlets (F = 1), (ii) hyper- proportional outlets (F > 1), and (iii) subproportional outlets (F < 1). When a certain change in the distributary discharge causes a proportionate change in the outlet discharge, the outlet (or semi-module) is said to be proportional. A proportional semi-module ensures proportionate distribution of water when the distributary discharge cannot be kept constant. For a proportionalsemi-modular oulet (F = 1),

$$\frac{H}{h} = \frac{m}{n} \qquad (5.6)$$

The ratio (H/h) is a measure of the location of the outlet and is termed setting. Every semi-module can work as a proportional semi-module if its sill is fixed at a particular level with respect to the bed level of the distributary. A semi-module set to behave as a proportional outlet may not remain proportional at all distributary discharges. Due to silting in the head reach of a distributary, the water level in the distributary would rise and the outlet located in the head reach would draw more discharge although the distributary discharge has not changed. Semi-modules of low flexibility are least affected by channel discharge and channel regimeand should, therefore, be used whenever the modular outlet is unsuitable for given site conditions.

The setting for a proportional outlet is equal to the ratio of the outlet and the channel indices. For hyper-proportional and sub-proportional outlets the setting must be, respectively, less and more than m/n. For a wide trapezoidal (or rectangular) channel, n can be approximately taken as 5/3 and for an orifice type outlet, m can be taken as 1/2. Thus, an orifice-type module will be proportional if the setting (H/h) is equal to (1/2)/(5/3), i.e., 0.3. The module will be hyper-proportional if the setting is less than 0.3 and sub-proportional if the setting is greater than 0.3. Similarly, a free flow weir type outlet (m = 3/2) would be proportional when the setting equals 0.9 which means that the outlet is fixed at 0.9 h below the water surface in the distributary.

Sensitivity

The ratio of the rate of change of discharge (dQ_0/Q_0) of an outlet to the rate of change in the water surface level of the distributary channel with respect to the depth of flow in the channel is called the 'sensitivity' of the outlet. Thus,

$$S = \frac{(dQ_0/Q_0)}{(dG/h)}$$

(5.7)

Here, S is the sensitivity and G is the gauge reading of a gauge which is so set that G = 0 corresponds to the condition of no discharge through the outlet (i.e., $Q_0 = 0$). Obviously, dG = dh. Thus, sensitivity can also be defined as the ratio of the rate of change of discharge of an outlet to the rate of change of depth of flow in the distributary channel. Therefore,

$$S = (dQ_0/Q_0)/(dh/h)$$

Also, $$F = (dQ_0/Q_0)/(dQ/Q)$$

$$= (dQ_0/Q_0)/n\left(\frac{dh}{h}\right)$$

$$= \frac{1}{n} S$$

$$\therefore \quad S = nF \quad (5.8)$$

Thus, the sensitivity of an outlet for a wide trapezoidal (or rectangular) distributary channel (n = 5/3) is equal to (5/3)F. The sensitivity of a modular outlet is, obviously, zero.

The 'minimum modular head' is the minimum head required for the proper functioning of the outlet as per its design. The modular limits are the extreme values of any parameter (or quantity) beyond which an outlet is incapable of functioning according to its design. The modular range is the range (between modular limits) of values of a quantity within which the outlet works as per its design. The efficiency of any outlet is equal to the ratio of the head recovered (or the residual head after the losses in the outlet) to the input head of the water flowing through the outlet.

Non-Modular Outlets

The non-modular outlet is usually in the form of a submerged pipe outlet or a masonry sluice which is fixed in the canal bank at right angles to the direction of flow in the distributary. The diameter of the pipe varies from 10 to 30 cm. The pipe is laid on a light concrete foundation to avoid uneven settlement of the pipe and consequent leakage problems. The pipe inlet is generally kept about 25 cm below the water level in the distributary. When considerable fluctuation in the distributary water level is anticipated, the inlet is so fixed that it is below the minimum water level in the distributary. Figure 5.4 shows a pipe outlet. If H is the difference in water levels of the distributary and the watercourse then the discharge Q through the outlet can be obtained from the equation,

$$H = \frac{V^2}{2g}\left(0.5 + \frac{fL}{d} + 1\right) \quad (5.9)$$

or $$H = \frac{V^2}{2g}\left(1.5 + f\frac{L}{d}\right) \quad (5.10)$$

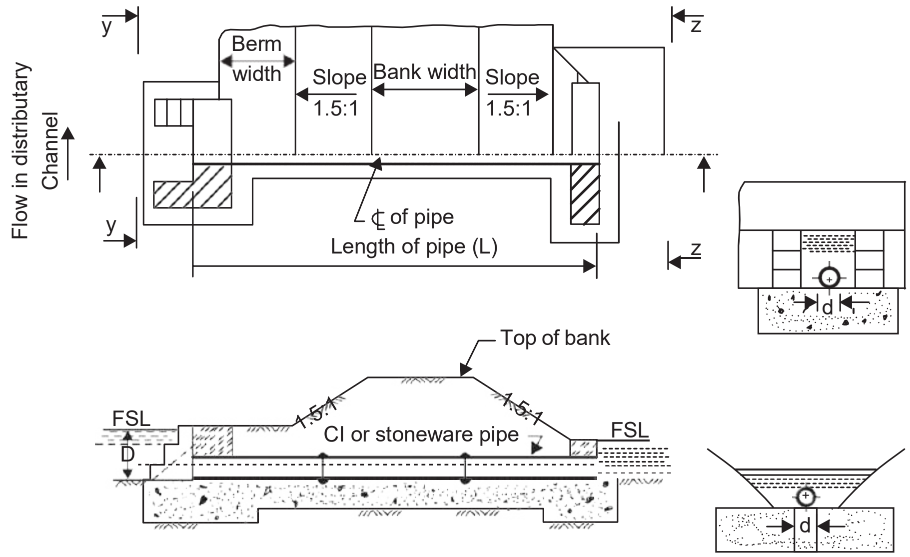
where $V = \dfrac{Q}{(\pi/4)\,d^2}$

$\sqrt{2gH}$

$$\left(\frac{d}{1.5d + fL}\right)^{1/2} \tag{5.11}$$

d = diameter of pipe outlet
L = length of pipe outlet
and f = friction factor for pipe.

x



Plan

Section yy

Section xx **Section zz**

Fig. 5.4 Pipe outlet (3)

Alternatively, the discharge Q can be expressed as

$$Q = AV$$

$$= \frac{\pi}{4} d^2 \left(\frac{d}{1.5d + fL}\right)^{1/2} \sqrt{2gH}$$

or $$Q = CA\sqrt{2gH} \qquad (5.12)$$

in which $$C = \left(\frac{d}{1.5d + fL}\right)^{1/2}$$

Because of the disturbance at the entrance, the outlet generally carries its due share of sediment. In order to further increase the amount of sediment drawn by the outlet, the inlet end of the outlet is lowered. It is common practice to place the pipe at the bed of the distributary to enable the outlets to draw a fair share of sediment (3). The outlet pipe thus slopes upward. This arrangement increases the amount of sediment withdrawn by the outlet without affecting the discharge through the outlet.

Obviously, the discharge through non-modular outlets varies with water levels in the distributary and watercourse. In the case of fields located at high elevations, the watercourse level is high and, hence, the discharge is relatively small. But, for fields located at low elevations, the discharge is relatively large due to lower watercourse levels. Further, depending upon the amount of withdrawal of water in the head reaches, the tail reach may be completely dry or get flooded. Thus, discharge through pipe outlets can be increased by deepening the watercourse and thereby lowering the water level in it. The discharge varies from outlet to outlet because of flow conditions, and also at different times on the same outlet due to sediment discharge in the distributary channel. For these reasons, proper and equitable distribution of water is very difficult. These are the serious drawbacks of pipe outlets. The non-modular outlets can, however, work well for low heads too and this is their chief merit. Pipe outlets are adopted in the initial stages of distributions or for additional irrigation in a season when excess supply is available.

Semi-Modular Outlets (Semi-Modules or Flexible Outlets)

The simplest type of semi-modular outlet is a pipe outlet discharging freely into the atmosphere. The pipe outlet, described as the non-modular outlet, works as semi-module when it discharges freely into the watercourse. The exit end of the pipe is placed higher than the water level in the watercourse. In this case, the working head H is the difference between the water level in the distributary and the centre of the pipe outlet. The efficiency of the pipe outlet is high and its sediment conduction is also good. The discharge through the pipe outlet cannot be increased by the cultivator by digging the watercourse and thus lowering the water level of the watercourse. Usually, a pipe outlet is set so that it behaves as subproportional outlet, i.e., its setting is kept less than 0.3. Other types of flexible outlets include Kennedy's gauge outlet, open flume outlet, and orifice semi-modules.

Kennedy's Gauge Outlet

This outlet was developed by RG Kennedy in 1906. It mainly consists of an orifice with bellmouth entry, a long expanding delivery pipe, and an intervening vertical air column above the throat (Fig. 5.5). The air vent pipe permits free circulation of air around the jet. This arrangement makes the discharge through the outlet independent of the water level in the watercourse. The water jet enters the cast iron expanding pipe which is about 3 m long and at the end of which a cement concrete pipe extension is generally provided. Water is then discharged into the watercourse. This outlet can be easily tampered with by the cultivator who blocks the air vent pipe to increase the discharge through the outlet. Because of this drawback and its high cost, Kennedy's gauge outlet is generally not used.

Open Flume Outlet

An open flume outlet is a weir with a sufficiently constricted throat to ensure supercritical flow and long enough to ensure that the controlling section remains within the throat at all discharges up to the maximum. A gradual expansion is provided downstream of the throat. The entire structure is built in brick masonry but the controlling section is generally provided with cast iron or steel bed and check plates. This arrangement ensures the formation of hydraulic jump and, hence, the outlet discharge remains independent of the water level in the watercourse. Figure 5.6 shows the type of open flume outlet commonly used in Punjab. The discharge through the outlet is proportional to $H^{3/2}$. The efficiency of the outlet varies between 80 and 90 per cent.

The throat width of the outlet should not be less than 60 mm as a narrower throat may easily get blocked by the floating material. For the range of outlet discharges normally used, the outlet is either deep and narrow, or shallow and wide. While a narrow outlet gets easily blocked, a shallow outlet is not able to draw its fair share of sediment.

Orifice Semi-Modules

An orifice semi-module consists of an orifice followed by a gradually expanding flume on the downstream side (Fig. 5.7). Supercritical flow through the orifice causes the formation of hydraulic jump in the expanding flume and, hence, the outlet discharge remains independent of the water level in the watercourse. The roof block is suitably shaped to ensure converging streamlines so that the discharge coefficient does not vary much. The roof block is fixed in its place by means of two bolts embedded in a masonry key. For adjustment, this masonry can be dismantled and the roof block is suitably adjusted. After this, the masonry key is rebuilt. Thus, the adjustment can be made at a small cost. Tampering with the outlet by the cultivators would be easily noticed through the damage to the masonry key. This is the chief merit of this outlet.

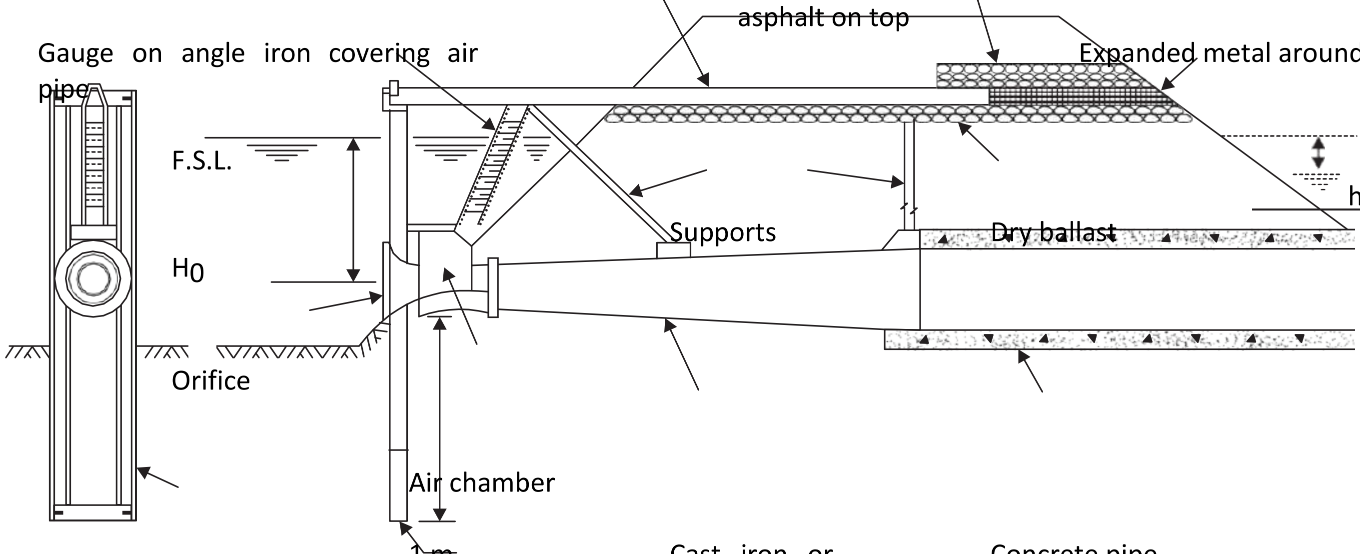
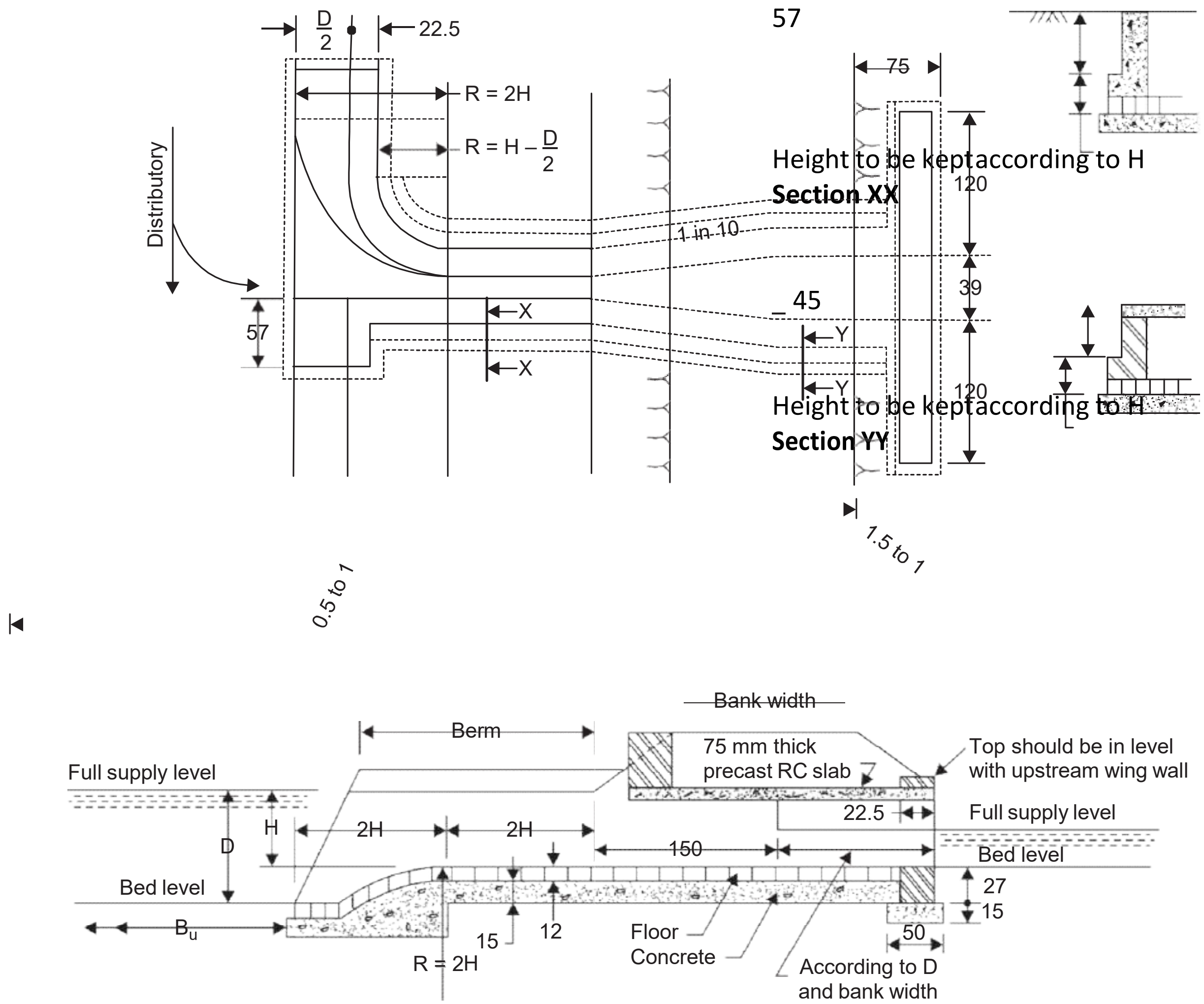
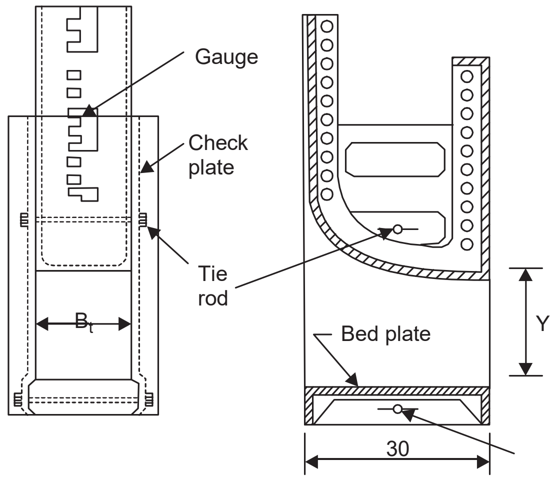


Fig. 5.5 Kennedy’s gauge outlet



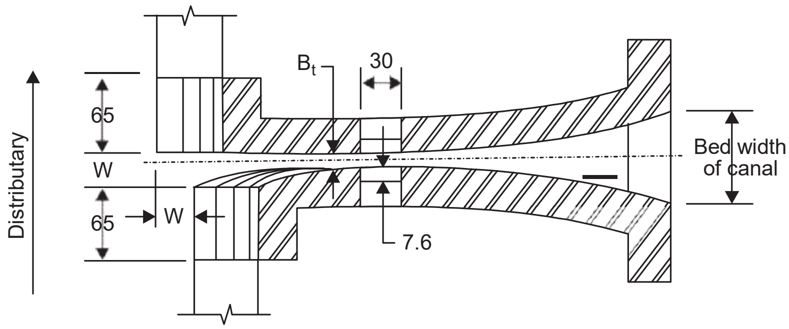
All dimensions in centimetres

Fig. 5.6 Plan of open flume outlet for distributary above 0.6 m depth and H less than full supply depth (3)



Tie rod

Details of roof block



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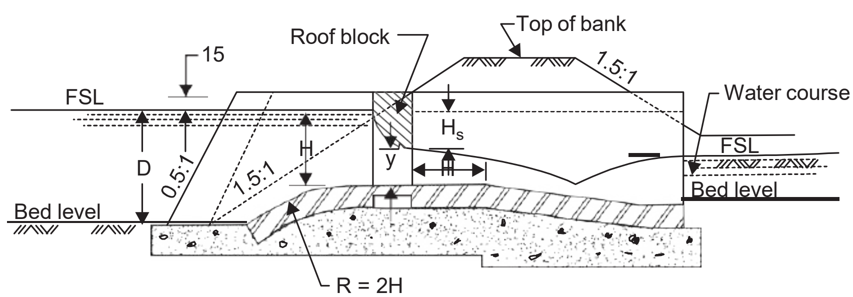


Longitudinal section

All dimensions in centimetres

Fig. 5.7 Crump's adjustable proportional module (3)

The base plates and roof blocks are manufactured in standard sizes, such as B_t = 6.1, 7.6, 9.9, 12.2, 15.4, 19.5, 24.4, and 30.5 cm. B_t is the throat width. The base plates and roof blocks of these standard sizes with required opening of the orifice are used to obtain desiredsupply through the outlet.

The waterway in this type of outlet is either deep and narrow, that can easily get blocked, or shallow and wide in which case it does not draw its fair share of sediment. The discharge in this type of outlet is given by the formula (3):

$Q = 4.03\ B_t\ Y$ (5.13)

The ratio H_s/D should be between 0.375 and $\sqrt{H}$0.48 for proportionate distribution of sediment and should be 0.8 or less for modular working (3). Here, H_s, D (= h), B_t, Y, and H areas shown in Fig. 5.7.

Modular Outlets

Most of the modular outlets have moving parts which make them costly to install as well as maintain. The following two types of modular outlets (also known as rigid modules), however, do not have any moving part:

Gibb's rigid module, and

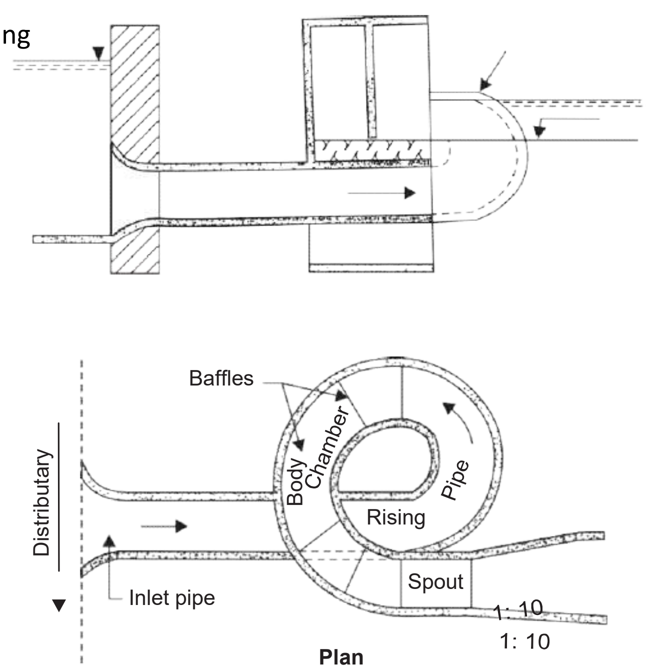
Khanna's rigid module.

Gibb's Rigid Module

This module has an inlet pipe under the distributary bank. This pipe takes water from distributary to a rising spiral pipe which joins the eddy chamber (Fig. 5.8). This arrangement results in free vortex motion. Due to this free vortex motion, there is heading up of water (owing to smaller velocity at larger radius–a characteristic of vortex motion) near the outer wall of the rising pipe. The water surface thus slopes towards the inner wall. A number of baffle plates of suitable size are suspended from the roof of the eddy chamber such that the lower ends of these plates slope against the flow direction. With the increase in head, the water banks up at the outer wall of the eddy chamber and impinges against the baffles and spins round in the compartment between two successive baffle plates. This causes dissipationof excess energy and results in constant discharge. The outlet is relatively more costly and itssediment withdrawal is also not good.

F.S.L. Curved rising pipe

Watercoursebed



Section

Fig. 5.8 Gibb's module

Khanna's Rigid Orifice Module

This outlet is similar to an orifice semi-module. But, in addition, it has sloping shoots fixed in the roof block (Fig. 5.9). These shoots cause back flow and thus keep the outlet discharge constant. If the water level in the distributary is at or below its normal level, the outlet behaves like an orifice semi-module. But, when the water level in the distributary channel is above its normal level, the water level rises in chamber A, and enters the first sloping shoot. This causes back flow and dissipates additional energy. This maintains a constant discharge. The number of sloping shoots and their height above the normal level can vary to suit local requirements. The shoots are housed in a chamber to prevent them from being tampered with. If the shoots are blocked, the outlet continues to function as a semi-module.

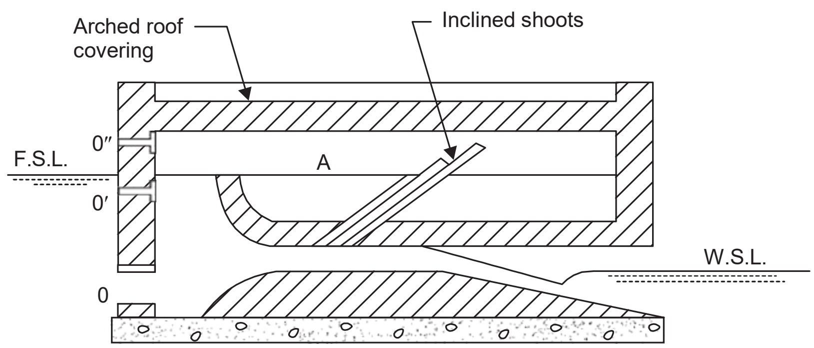


Fig. 5.9 Khanna's module

Example 5.2 A semi-modular pipe outlet of diameter 15 cm is to be installed on a distributary with its bed level and full supply level at 100.3 and 101.5 m, respectively. The maximum water level in the watercourse is at 101.15 m. Set the outlet for maximum discharge and calculate the same. The coefficient C in the discharge equation, Eq. (5.12), may be taken as 0.62. Is the setting proportional, subproportional or hyper-proportional?

Solution: For maximum discharge, the pipe outlet must be at the maximum water level in the watercourse.

Therefore,

$$H = (101.5 - 101.15) - \frac{0.15}{2} = 0.275 \text{ m}$$

and $\quad Q = CA\sqrt{2gH}$

$$= 0.62 \times \frac{3.14 \times (0.15)^2}{4}\sqrt{2 \times 9.81 \times 0.275}$$

$= 0.0254$ m^3/s

Flexibility $\quad F = \frac{m}{n}\frac{h}{H}$

and $\quad \frac{m}{n} = \frac{1}{2} \times \frac{3}{5} = 0.3$

Therefore, $\quad F = \frac{0.3 \times 1.2}{0.275} = 1.309$

Therefore, the setting is hyper-proportional.

CANAL REGULATION

The amount of water which can be directed from a river into the main canal depends on: (i) the water available in the river, (ii) the canal capacity, and (iii) the share of other canals taking off from the river. The flow in the main canal is diverted to various branches and distributaries.

The distribution of flow, obviously, depends on the water demand of various channels. The method of distribution of available supplies is termed canal regulation.

When there exists a significant demand for water anywhere in the command area of a canal, the canal has to be kept flowing. The canal can, however, be closed if the water demand falls below a specified quantity. It is reopened when the water demand exceeds the specified minimum quantity. Normally, there always exists a demand in some part of the command area of any major canal. Such major canals can, therefore, be closed only for a very small period (say, three to four weeks in a year). These canals run almost continuously and carry discharges much less than their full capacity, either when there is less demand or when the available supplies are insufficient.

If the demand is less, only the distributaries which need water are kept running and the others (including those which have very little demand) are closed. In case of keen demand, but insufficient supplies, either all smaller channels run simultaneously and continuously with reduced supplies, or some channels are closed turn by turn and the remaining ones run with their full or near-full capacities. The first alternative causes channel silting, weed growth, increased seepage, waterlogging, and low heads on outlets. The second alternative does not have these disadvantages and allows sufficient time for inspection and repair of the channels.

A roster is usually prepared for indicating the allotted supplies to different channels and schedule of closure and running of these channels. It is advantageous to have flexible regulation so that the supplies can be allocated in accordance with the anticipated demand. The allocation of supplies is decided on the basis of the information provided by the canal revenue staff who keep a close watch on the crop condition and irrigation water demand.

The discharge in canal is usually regulated at the head regulator which is usually designed as a meter. When the head regulator cannot be used as discharge meter, a depth gauge is provided at about 200 m downstream of the head regulator. The gauge reading is suitably related to the discharge. By manipulating the head regulator gates, the desired gauge reading (and, hence, the discharge) can be obtained.

DELIVERY OF WATER TO FARMS

Once water has been brought to the watercourse, the problem of its equitable distribution amongst the farms located along the watercourse arises. There are the following two possible alternatives (4), each with its own merit, for achieving this objective:

Restrict the canal irrigation to such limited areas as can be fully supported with the lowest available supply. This does not lead to the total utilisation of available water. Agricultural production and protection against famine would also not be optimum. The production would be maximum per unit of land covered though not per unit of water available. It would, however, not require a precise or sophisticated method for the distribution of irrigation water. The delivery system for this alternative can be either continuous or demand-based, depending upon the availability of water. A continuous delivery system can be effectively used for large farms and continuously terraced rice fields. Though ideal, a demand-based delivery system is not practical on large irrigation systems.

Extend irrigation to a much larger area than could be supported by the lowest available supply. This creates perpetual scarcity of irrigation water but ensures that a comparatively less quantity of water remains unutilised. Agricultural production and the protection against famine would be at the optimum levels. The production would be maximum per unit of available water though not per unit of land covered. This method would have greater social appeal, and requires precise and sophisticated methods for equitable distribution of irrigation water.

Irrigation water from a distributary can possibly be delivered to farmers in the following four different ways:

Continuous delivery system

Free demand system

Rotation delivery system

Controlled demand (or modified rotation) system.

In the continuous delivery system water is supplied continuously to the farm at a predetermined rate. This system is easy to operate, but would generally result in excessive water applications. This delivery system can be efficient only if the farms are so large that the farmer can redistribute his supply at the farm to different pockets of his farm in accordance with crop and soil conditions. This may necessitate regulatory storage at the farm for efficient utilization of the water delivered to the farm. Large corporation farms or state-owned farms can be served efficiently by this system.

In a free demand system, farmers take intermittent delivery at will, depending upon the needs of their crops, from the constantly available supply in such a manner that their instantaneous withdrawal rate does not exceed that for which they subscribe and which also corresponds, in some way to the installed capacity. Obviously, this system provides maximum flexibility but requires that the farmer is closely aware of crop irrigation requirement and does not have tendency to overirrigate when water is not sold by volume. This system also leads to uncontrolled peak demands during daylight hours and excessive operational losses during night. The day-time peak demands may require large delivery capabilities. Free demand system is well adapted to well irrigation rather than canal irrigation.

In the rotation delivery system the canal authority assumes responsibility for allocating the continuous flow available in the relevant distributary to each farmer of the area which is served by the distributary. The farmers get water according to a fixed delivery schedule. This method is capable of achieving equitable distribution to a large number of farmers with relatively lesser water supplies. Hence, this method is generally adopted for canal irrigation supplies in India and is known by the name of 'warabandi' in India and has been described in greater detail in the next article.

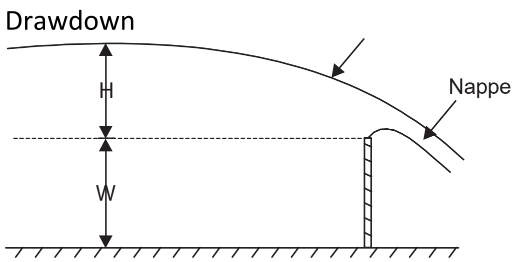
Obligatory use of water supplied to the farm may cause considerable wastage of water and result in waterlogging without increasing the production. On the other hand, failure to adjust rotation schedules to crop irrigation requirements may result in inefficient irrigation during crucial growth periods and thus affect adversely the crop production. To overcome the deficiency of the rotation delivery method, water may, alternately, be delivered according to some kind of controlled demand system which is a sort of compromise between free demand and rotation delivery systems. In this modified system of modified rotation, priority for delivery is on a rotation basis, but actual delivery may deviate depending upon the actual demand. Obviously, much better coordination between farmers and the authorities would be required for this system to work efficiently.

FLOW MEASUREMENT

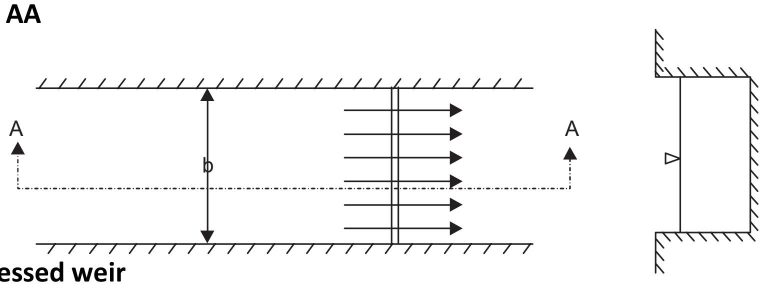
The importance of accurate flow measurement for proper regulation, distribution, and charging of irrigation water cannot be over emphasized. There are several flow measuring devices available for flow measurement in irrigation systems. Generally weirs and flumes are used for this purpose. Besides, there are some other indirect methods by which velocities are measured and the discharge computed. In these methods, the channel section is divided into a suitable number of compartments and the mean velocity of flow for each of these compartments is measured by using devices such as current meter, surface floats, double floats, velocity rods, and so on. The discharge through any compartment is obtained by multiplying the mean velocity of flow with the area of cross-section of the compartment. The sum of all compartmental discharges gives the channel discharge.

Weirs

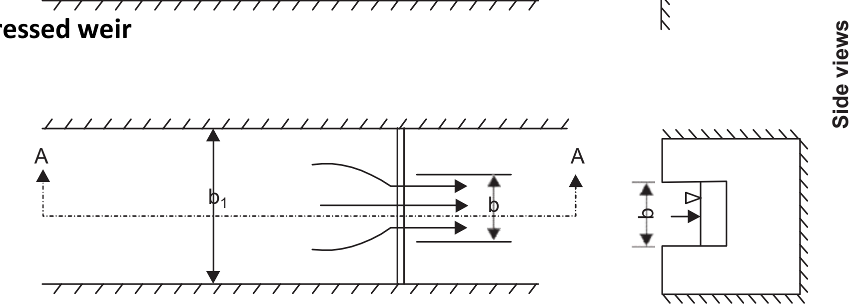
Weirs have been in use as discharge measuring devices in open channels for almost two centuries. A weir is an obstruction over which flow of a liquid occurs (Fig. 5.11). Head H over the weir is related to the discharge flowing and, hence, the weir forms a useful discharge measuring device. Weirs can be broadly classified as thin-plate (or sharp-crested) and broad-crested weirs.



Section along AA



Plan of suppressed weir



Plan of contracted weir

Fig. 5.11 Flow over suppressed and contracted weirs

Thin-Plate Weirs

A sharp-crested (or thin-plate) weir is formed in a smooth, plane, and vertical plate and its edges are bevelled on the downstream side to give minimum contact with the liquid. The area of flow is most commonly either triangular or rectangular and, accordingly, the weir is said to be a triangular or rectangular weir. In general, the triangular weir (or simply the V-notch) is used for the measurement of low discharges, and the rectangular weir for the measurement of large discharges.

The pattern of the flow over a thin-plate weir is very complex and cannot be analysed theoretically. This is due to the non-hydrostatic pressure variation (on account of curvature of streamlines), turbulence and frictional effects, and the approach flow conditions. The effects of viscosity and surface tension also become important at low heads. Therefore, the analytical relation (between the rate of flow and the head over the weir), obtained after some simplifying assumptions, are suitably modified by experimentally determined coefficients. Following this approach, Ranga Raju and Asawa (5) obtained the following discharge equations:

For thin-plate triangular weir with notch angle θ

$$Q = k_1 \frac{8}{15} C_d \sqrt{2g} \, (\tan \theta/2) \, H^{5/2} \qquad (5.14)$$

For a suppressed thin-plate rectangular weir

$$Q = k_1 \left[\frac{2}{3} \left(0.611 + 0.075 \frac{H}{P} \right) b \sqrt{2g} \, H^{3/2} \right] \qquad (5.15)$$

N3 H W K Q

where, Q = discharge flowing over the weir,

H = head over the weir,

b = width of the weir,

C_d = coefficient of discharge for triangular weir (Fig. 5.12),

A = area of cross-section of the approach flow,

k_1 = correction factor to account for the effects of viscosity and surface tension(Fig. 5.13),

R_e = $g^{1/2}$ $H^{3/2}$/? (typical Reynolds number),

? = kinematic viscosity of the flowing liquid,

W_1 = ?gH^2/? (typical Weber number),

? = surface tension of the flowing liquid,

? = mass density of the flowing liquid, and

g = acceleration due to gravity.

It should be noted that k_1 = 1.0 for $R_e^{0.2}$ $W_1^{0.6}$ greater than 900. This limit corresponds to a head of 11.0 cm for water at 20°C. The mean line drawn in Fig. 5.13 can be used to find the

value of k_1. The scatter of data (not shown in the figure) was generally less than 5 per cent implying maximum error of ± 5 per cent in the prediction of discharge.

Equation (5.15) along with Fig. 5.13, and Eq. (5.14) along with Figs. 5.12 and 5.13 enable computations of discharge over a suppressed thin-plate rectangular weir and a thin-plate 90°-triangular weir, respectively. A weir is termed suppressed when its width equals the channel width and in such cases the ventilation of nappe becomes essential.

Broad-Crested Weirs

Broad-crested weirs are generally used as diversion and metering structures in irrigation systems in India. The weir has a broad horizontal crest raised sufficiently above the bed so that the cross-sectional area of the approaching flow is much larger than the cross- sectional area of flow over the top of the weir. The upstream edge of the weir is well rounded to avoid undue eddy formation and consequent loss of energy. The derivation of the discharge equation for flow over a broad-crested weir is based on the concept of critical flow.

The advantages of weirs for discharge measurement are as follows:

Simplicity and ease in construction,

Durability, and

Accuracy.

However, the main requirement of considerable fall of water surface makes their use in areas of level ground impracticable. Besides, deposition of sand, gravel, and silt upstream of the weir prevents accurate measurements.

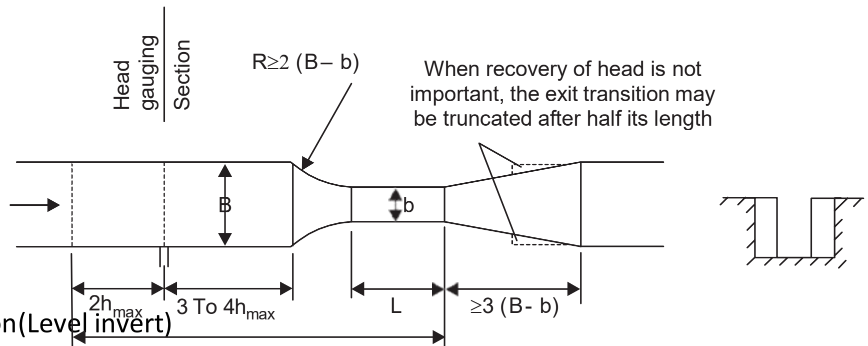
Flumes

A flume is a flow measuring device formed by a constriction in an open channel. The constrictioncan be either a narrowing of the channel or a narrowing in combination with a hump in the invert. By providing sufficient amount of constriction, it is possible to produce critical flow conditions there. When this happens, there exists a unique stage-discharge relationship independent of the downstream conditions. The use of critical-depth flumes for discharge measurement is based on this principle.

The main advantage of a critical-depth flume over a weir is in situations when material (sediment or sewage) is being transported by the flow. This material gets deposited upstream of the weir and affects the discharge relation and results in a foul-smelling site in case of sewage flow. The critical-depth flumes consisting only of horizontal contraction would easily carry the material through the flume. Critical-depth fumes can be grouped into two main categories.

Long-Throated Flumes

The constriction of these flumes (Fig. 5.22) is sufficiently long (the length of the throat should be at least twice the maximum head of water that will occur upstream of the flume) so that it produces small curvature in the water surface and the flow in the throat is virtually parallel to the invert of the flume.

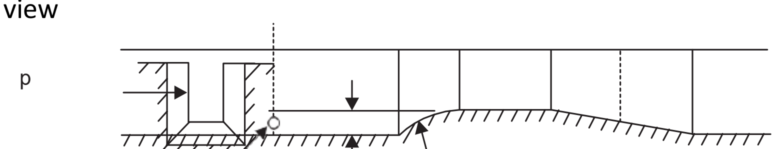


Front elevation(Level invert)

In a flume without a hump (p = 0),

the invert over this length shall be truly level

Plan view



Connection to stilling well This radius is chosen so that bottom contraction starts at the same section as side contractions. For flume with bottom contraction only, radius = 4p

Front elevation (With hump)

Fig. 5.22 Geometry of rectangular long-throated flume

This condition results in nearly hydrostatic pressure distribution at the control section (where critical depth occurs) which, in turn, allows analytical derivation of the stage-discharge relation. This gives the designer the freedom to vary the dimensions of the flume in order to meet specific requirements. Such flumes are usually of rectangular, trapezoidal, triangular or U-shaped cross-section. For a rectangular flume, the discharge of an ideal fluid is expressed as

$$Q = \left(\frac{2}{3}\right)^{3/2} \sqrt{g}\, bH^{3/2} \qquad (5.21)$$

Here, H represents the upstream energy and b is the typical width dimension for the particular cross-sectional shape of the flume. By introducing suitable coefficients this equation can be generalised in the following form so that it applies to any cross-sectional shape (9):

$$Q = \left(\frac{2}{3}\right)^{3/2} \sqrt{g}\, C_v C_s C_d\, bh^{3/2} \qquad (5.22)$$

where, C_v = coefficient to take into account the velocity head in the approach channel,

C_s = coefficient to take account of the cross-sectional shape of the flume,

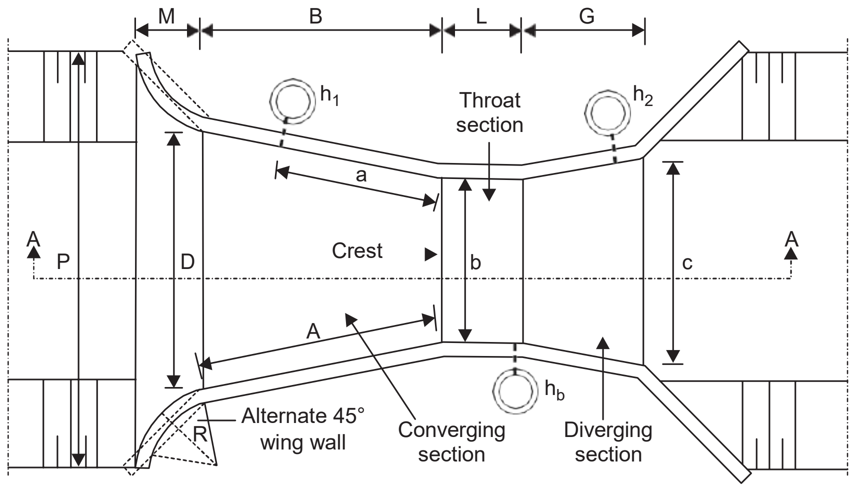
C_d = coefficient for energy loss,

and h = depth of water, upstream of the flume, measured relative to the invert level of the throat (i.e., gauged head).

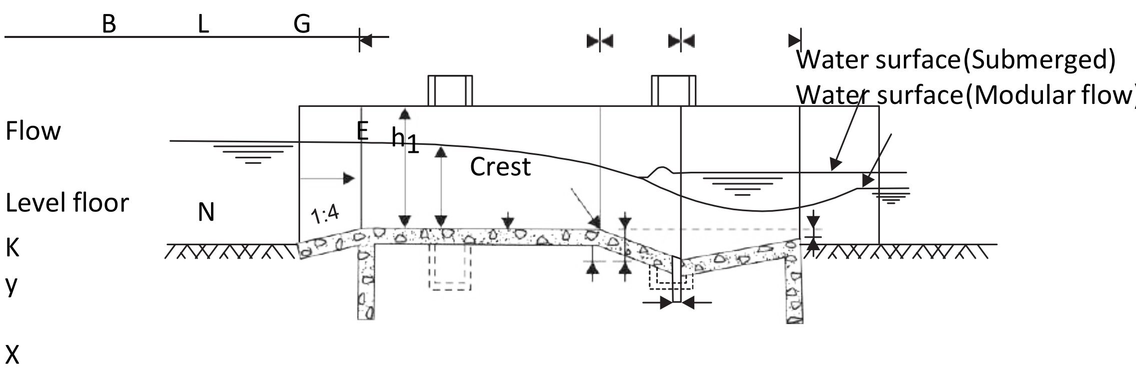
Short-Throated Flumes

In these flumes, the curvature of the water surface is large and the flow in the throat is not parallel to the invert of the flume. The principle of operation of these flumes is the same as that of long-throated flumes, viz. the creation of critical conditions at the throat. However, non-hydrostatic pressure distribution (due to large curvature of flow) does not permit analytical derivation of the discharge equation. Further, energy loss also cannot be assessed. Therefore, it becomes necessary to rely on direct calibration either in the field or in the laboratory for the determination of the discharge equation. The designer does not have complete freedom in choosing the dimensions of the flume but has to select the closest standard design to meet his requirements. Such flumes, however, require lesser length and, hence, are more economical than long-throated flumes. One of the most commonly used short-throated flumes is the Parshallflume which has been described here.

Parshall (10, 11 and 12) designed a short-throated flume with a depressed bottom (Fig. 5.23) which is now known as the Parshall flume. This was first developed in the 1920's in the USA and has given satisfactory service at water treatment plants and irrigation projects. It consists of short parallel throat preceded by a uniformly converging section and followed by a uniformly expanding section. The floor is horizontal in the converging section, slopes downwards in the throat, and is inclined upwards in the expanding section. The control section at which the depth is critical, occurs near the downstream end of the contraction.



Plan



Section A-A

Fig. 5.23 Parshall flume

There are 22 standard designs covering a wide range of discharge from 0.1 litre per second to 93 m^3/s.

Current Meter

The current meter is a widely used mechanical device for the measurement of flow velocity and, hence, the discharge in an open channel flow. It consists of a small wheel with cups at the periphery or propeller blades rotated by the force of the flowing water, and a tail or fins to keep the instrument aligned in the direction of flow. The cup-type current meter has a vertical axis, and is a more rugged instrument which can be handled by relatively unskilled technicians. The propeller-type current meter has been used for relatively higher velocities (up to 6 to 9 m/ s as against 3 to 5 m/s for the cup-type current meter). The small size of the propeller-type current meter is advantageous when the measurements have to be taken close to the wall. The propeller-type meter is less likely to be affected by floating weeds and debris.

For measurements, the current meter is mounted on a rod and moved vertically to measure the velocity at different points. The speed of rotation of cups or blades depends on the velocity of flow. The instrument has an automatic counter with which the number of rotationsin a given duration is determined.

The current meter is calibrated by moving it with a known speed in still water and noting the number of revolutions per unit of time. During measurement, the current meter isheld stationary in running water. Using the appropriate calibration (supplied by the manufacturer) the velocity can be predicted. By this method one can obtain velocity distribution and, hence, the discharge. Or, alternatively, one can measure the velocity at 0.2 h and 0.8 h (here, h is the depth of flow) below the free surface and the mean of the two values gives the average velocity of flow. Sometimes, velocity at 0.6 h is taken as the average velocity of flow

Other Methods

Mean velocities in open channels can, alternatively, be determined by measuring surface velocities using surface floats. The surface float is an easily visible object lighter than water, but sufficiently heavy not to be affected by wind. The surface velocity is measured by noting down the time the surface float takes in covering a specified distance which is generally not less than 30 metres and 15 metres for large and small channels, respectively. The surface velocity is multiplied by a suitable coefficient (less than unity) to get the average velocity of flow.

A double float consists of a surface float to which is attached a hollow metallic sphere heavier than water. Obviously, the observed velocity of the double float would be the mean of the surface velocity and the velocity at the level of the metallic sphere. By adjusting the metallic sphere at a depth nearly equal to 0.2 h above the bed, the observed velocity will be approximatelyequal to the mean velocity of flow.

Alternatively, velocity rods can be used for the measurement of average velocity of flow. Velocity rods are straight wooden rods or hollow tin tubes of 25 mm to 50 mm diameter and weighted down at the bottom so that these remain vertical and fully immersed except for a small portion at the top while moving in running water. These rods are either telescopic-type or are available in varying lengths so that they can be used for different depths of flow. As the rod floats vertically from the surface to very near the bed, its observed velocity equals the mean velocity of flow in that vertical plane.

For measuring discharge in a pipeline, one may employ either orifice meter or venturi meter or bend meter or any other suitable method.

ASSESSMENT OF CHARGES FOR IRRIGATION WATER

Irrigation projects involve huge expenditure for their construction. The operation and maintenance of these projects also require finances. With the introduction of irrigation facilities in an area, farmers of the area are immensely benefited. Hence, it is only appropriate that they are suitably charged for the irrigation water supplied to them.

The assessment of irrigation water charges can be done in one of the following ways:

Assessment on area basis,

Volumetric assessment,

Assessment based on outlet capacity,

Permanent assessment, and

Consolidated assessment.

In the area basis method of assessment, water charges are fixed per unit area of land irrigated for each of the crops grown. The rates of water charges depend on the cash value of crop, water requirement of crop, and the time of water demand with respect to the available supplies in the source. Since the water charges are not related to the actual quantity of water used, the farmers (particularly those whose holdings are in the head reaches of the canal) tend to overirrigate their land. This results in uneconomical use of available irrigation water besides depriving the cultivators in the tail reaches of the canal of their due share of irrigation water. However, this method of assessment, being simple and convenient, is generally used for almost all irrigation projects in India.

In volumetric assessment, the charges are in proportion to the actual amount of water received by the cultivator. This method, therefore, requires installation of water meters at all the outlets of the irrigation system. Alternatively, modular outlets may be provided to supply a specified discharge of water. This method results in economical use of irrigation water and is, therefore, an ideal method of assessment. However, it has several drawbacks. This method requires the installation and maintenance of suitable devices for measurement of water supplied. These devices require adequate head at the outlet. Further, there is a possibility of water theft by cutting of banks or siphoning over the bank through a flexible hose pipe. Also, the distribution of charges among the farmers, whose holdings are served by a common outlet, may be difficult. Because of these drawbacks, this method has not been adopted in India.

The assessment of canal water charges based on outlet capacity is a simple method and is workable if the outlets are rigid or semi-modular and the channel may run within their modular range.

In some regions, artificial irrigation, though not essential, has been provided to meet the water demand only in drought years. Every farmer of such a region has to pay a fixed amount. The farmers have to pay these charges even for the years for which they do not take any water. A farmer has also to pay a tax on the land owned by him. In the consolidated assessment method, both the land revenue and the water charges are combined and the cultivators are accordingly charged.

WATERLOGGING

In all surface water irrigation schemes, supplying the full water requirements of a crop, more water is added to the soil than is actually required to make up the deficit in the soil resulting from continuous evapotranspiration by crops. This excess water and the water that seeps into the ground from reservoirs, canals, and watercourses percolate deep into the ground to join the water table and, thus, raise the water table of the area. When the rising water table reaches the root zone, the pore spaces of the root-zone soil get saturated.

A land is said to be waterlogged when the pores of soil within the root zone of a plant gets saturated and the normal growth of the plant is adversely affected due to insufficient air circulation. The depth of the water table at which it starts affecting the plant would depend on plant and soil characteristics. A land would become waterlogged sooner for deep-rooted plants than for shallow-rooted plants. Impermeable soils generally have higher capillary rise and, hence, are waterlogged more easily than permeable soils. A land is generally waterlogged when the ground water table is within 1.5 to 2.0 m below the ground surface. Water table depth is good if the water table is below 2 m and rises to 1.8 m for a period not exceeding 30 days in a year (13). If the water table is at about 1.8 m and rises to about 1.2 m for a period not exceeding 30 days in a year, the condition is considered as fair. If the water table depth is between 1.2 to 1.8 m which may rise to 0.9 m for a period not exceeding 30 days in a year, the condition of water table depth is rather poor. In a poor condition of water table depth, the water level is less than 1.2 m from the surface and is generally rising.

A high water table increases the moisture content of the unsaturated surface soil and thus increases the permeability. There may be advantages of having water table close to the surface as it may result in higher crop yield due to favourable moisture supply. This may, however, be true only for few years after water table has risen from great depths. The favourable condition may be followed by serious decrease in the crop yield in areas where alkali salts are present. With slight increase in inflow to the ground, the high water table may become too close to the ground surface and when this happens the land gets waterlogged and becomes unsuitable for cultivation.

The problem of waterlogging is a world-wide phenomenon which occurs mainly due to the rise of the ground water table beyond permissible limits on account of the change in ground water balance brought about by the percolation of irrigation water. It has become a problem of great importance on account of the introduction of big irrigation projects. The land subjected to waterlogging results in reduction of agricultural production. The problem of waterlogging has already affected about 5 million hectares of culturable area in India (see Table 6.2 for more details).

Causes of Waterlogging

Ground water reservoirs receive their supplies through percolation of water from the ground surface. This water may be from rainfall, from lakes or water applied to the fields for irrigation. This water percolates down to the water table and, thus, raises its position. Depending upon the elevation and the gradient of the water table, the flow may either be from surface to the ground (i.e., inflow) or ground to the surface (i.e., outflow). Outflow from a ground water reservoir includes water withdrawn through wells and water used as consumptive use. An overall balance between the inflow and outflow of a ground water reservoir will keep the water table at almost fixed level. This balance is greatly disturbed by the introduction of a canal system or a well system for irrigation. While the former tends to raise the water table, the latter tends to lower it.

Waterlogging in any particular area is the result of several contributing factors. The main causes of waterlogging can be grouped into two categories: (i) natural, and (ii) artificial.

Natural Causes of Waterlogging

Topography, geological features, and rainfall characteristics of an area can be the natural causes of waterlogging.

In steep terrain, the water is drained out quickly and, hence, chances of waterlogging are relatively low. But in flat topography, the disposal of excess water is delayed and this water stands on the ground for a longer duration. This increases the percolation of water into the ground and the chances of waterlogging. The geological features of subsoil have considerable influence on waterlogging. If the top layer of the soil is underlain by an impervious stratum, the tendency of the area getting waterlogged increases.

Rainfall is the major contributing factor to the natural causes of waterlogging. Low- lying basins receiving excessive rainfall have a tendency to retain water for a longer period of time and, thus get, waterlogged. Submergence of lands during floods encourages the growth of weeds and marshy grasses which obstruct the drainage of water. This, again, increases the amount of percolation of water into the ground and the chances of waterlogging.

Artificial Causes of Waterlogging

There exists a natural balance between the inflow and outflow of a ground water reservoir. This balance is greatly disturbed due to the introduction of artificial irrigation facilities. The surface reservoir water and the canal water seeping into the ground increase the inflow to the ground water reservoir. This raises the water table and the area may become waterlogged. Besides, defective method of cultivation, defective irrigation practices, and blocking of naturaldrainage further add to the problem of water logging.

Effects of Waterlogging

The crop yield is considerably reduced in a waterlogged area due to the following adverseeffects of waterlogging:

Absence of soil aeration,
Difficulty in cultivation operations,
Weed growth, and
Accumulation of salts.

In addition, the increased dampness of the waterlogged area adversely affects the health of the persons living in that area.

Absence of Soil Aeration

In waterlogged lands, the soil pores within the root zone of crops are saturated and air circulationis cut off. Waterlogging, therefore, prevents free circulation of air in the root zone. Thus, waterlogging adversely affects the chemical processes and the bacterial activities which are essential for the proper growth of a plant. As a result, the yield of the crop is reduced considerably.

Difficulty in Cultivation

For optimum results in crop production, the land has to be prepared. The preparation of land (i.e., carrying out operations such as tillage, etc.) in wet condition is difficult and expensive. As a result, cultivation may be delayed and the crop yield adversely affected. The delayed arrival of the crop in the market brings less returns to the farmer.

Weed Growth

There are certain types of plants and grasses which grow rapidly in marshy lands. In waterlogged lands, these plants compete with the desired useful crop. Thus, the yield of the desired useful crop is adversely affected.

Accumulation of Salts

As a result of the high water table in waterlogged areas, there is an upward capillary flow of water to the land surface where water gets evaporated. The water moving upward brings with it soluble salts from salty soil layers well below the surface. These soluble salts carried by the upward moving water are left behind in the root zone when this water evaporates. The accumulation of these salts in the root zone of the soil may affect the crop yield considerably.

Remedial Measures for Waterlogging

The main cause of waterlogging in an area is the introduction of canal irrigation there. It is, therefore, better to plan the irrigation scheme in such a way that the land is prevented from getting waterlogged. Measures, such as controlling the intensity of irrigation, provision of intercepting drains, keeping the full supply level of channels as low as possible, encouraging economical use of water, removing obstructions in natural drainage, rotation of crops, running of canals by rotation, etc., can help considerably in preventing the area from getting waterlogged.

In areas where the water table is relatively high, canal irrigation schemes should be planned for relatively low intensity of irrigation. In such areas canal irrigation should be allowed in the Kharif season only. Rabi irrigation should be carried out using ground water. Intercepting drains provided along canals with high embankments collect the canal water seeping through the embankments and, thus, prevent the seeping water from entering the ground. The full supply level in the channels may be kept as low as possible to reduce the seepage losses. The level should, however, be high enough to permit flow irrigation for most of the command area of the channel. For every crop there is an optimum water requirement for the maximum yield. The farmers must be made aware that the excessive use of water would harm the crop rather than benefit it. The levelling of farm land for irrigation, and a more efficient irrigation system decrease percolation to the ground and reduce the chances of water logging. The improvement in the existing natural drainage would reduce the amount of surface water percolating into the ground. A judicious rotation of crops can also help in reducing the chances of water logging. Running of canals by rotation means that the canals are run for few days and then kept dry for some days. This means that there would not be seepage for those days when the canal is dry. This, of course, is feasible only in case of distributaries and watercourses.

The combined use of surface and subsurface water resources of a given area in a judicious manner to derive maximum benefits is called conjunctive use of water. During dry periods, the use of ground water is increased, and this results in lowering of the water table. The use of surface water is increased during the wet season. Because of the lowered water table, the ground water reservoir receives rainfall supplies through increased percolation. The utilisation of water resources in this manner results neither in excessive lowering of the water table nor in its excessive rising. The conjunctive use of surface and subsurface water serves as a precautionary measure against waterlogging. It helps in greater water conservation and lower evapotranspiration losses, and brings larger areas under irrigation.

The most effective method of preventing waterlogging in a canal irrigated area, however, is to eliminate or reduce the seepage of canal water into the ground. This can be achieved by the lining of irrigation channels (including watercourses, if feasible). In areas which have already become waterlogged, curative methods such as surface and subsurface drainage and pumping of ground water are useful.

Lining of Irrigation Channels

Most of the irrigation channels in India are earthen channels. The major advantage of an earth channel is its low initial cost. The disadvantages of an earth channel are: (i) the low velocity of flow maintained to prevent erosion necessitates larger cross-section of channels, (ii) excessive seepage loss which may result in waterlogging and related problems such as salinity of soils, expensive road maintenance, drainage activities, safety of foundation structures, etc.,

(iii) favourable conditions for weed growth which further retards the velocity, and (iv) the breaching of banks due to erosion and burrowing of animals. These problems of earth channels can be got rid of by lining the channel.

A lined channel decreases the seepage loss and, thus, reduces the chances of waterlogging. It also saves water which can be utilised for additional irrigation. A lined channel provides safety against breaches and prevents weed growth thereby reducing the annual maintenance cost of the channel. Because of relatively smooth surface of lining, a lined channel requires a flatter slope. This results in an increase in the command area. The increase in the useful head is advantageous in case of power channels also. The lining of watercourses in areas irrigated by tubewells assume special significance as the pumped water supply is more costly.

As far as practicable, lining should, however, be avoided on expansive clays (14). But, if the canal has to traverse a reach of expansive clay, the layer of expansive clay should be removed and replaced with a suitable non-expansive soil and compacted suitably. If the layer of expansive clay is too thick to be completely excavated then the expansive clay bed is removed to a depth of about 60 cm and filled to the grade of the underside of lining with good draining material. The excavated surface of expansive clay is given a coat of asphalt to prevent the entry of water into the clay.

The cost of lining a channel is, however, the only factor against lining. While canal lining provides a cost-effective means of minimising seepage losses, the lining itself may rapidly deteriorate and require recurring maintenance inputs if they are to be effective in controllingseepage loss. A detailed cost analysis is essential for determining the economic feasibility of lining a channel. The true cost of lining is its annual cost rather than the initial cost. The cost of lining is compared with the direct and indirect benefits of lining to determine the economic feasibility of lining a channel. Besides economic factors, there might be intangible factors suchas high population density, aesthetics, and so on which may influence the final decision regarding the lining of a channel.

Economics of Canal Lining

The economic viability of lining of a canal is decided on the basis of the ratio of additional benefits derived from the lining to additional cost incurred on account of lining. The ratio is worked out as follows (15):

Let C = cost of lining in Rs/sq. metre including the additional cost of dressing the banks for lining and accounting for the saving, if any, resulting from the smaller cross-sections and, hence, smaller area of land, quantity of earth work, and structures required for the lined sections. This saving will be available on new canals excavated to have lined cross-section right from the beginning, but not on lining of the existing unlined canals.

s and S = seepage losses in unlined and lined canals, respectively, in cubic metres per square metre of wetted surface per day of 24 hrs.

p and P = wetted perimeter in metres of unlined and lined sections, respectively,

T = total perimeter of lining in metres,

d = number of running days of the canal per year, W = value of water saved in rupees per cubic metre,L = length of the canal in metres,

y = life of the canal in years,

M = annual saving in rupees in operation and maintenance due to lining, taking into account the maintenance expenses on lining itself,

and B = annual estimated value in rupees of other benefits for the length of canal under consideration. These will include prevention of waterlogging, reduced cost of drainage for adjoining lands, reduced risk of breach, and so on.

The annual value of water lost by seepage from the unlined section

= pLsdW rupees.

The annual saving in value of water otherwise lost by seepage

= (pLsdW – PL SdW) rupees

= {LdW (ps – PS)} rupees

Total annual benefits resulting from the lining of canal

B_t = {LdW (ps – PS) + B + M} rupees (5.23) Additional capital expenditure on construction of lined canal

= TLC rupees

If the prevalent rate of interest is x per cent per year, the annual instalment a (rupees) required to be deposited each year (at its beginning) for a number of y years to amount to TLC plus its interest at the end of y years is determined by the following equation:

$$\text{TLC}\left(1+\frac{x}{100}\right)^{y} = a\left\{\left(1+\frac{x}{100}\right)^{y}+\left(1+\frac{x}{100}\right)^{y-1}+\left(1+\frac{x}{100}\right)^{y-2}+\ldots+\left(1+\frac{x}{100}\right)^{1}\right\}$$

$$= \frac{a\left(1+\frac{x}{100}\right)\left\{\left(1+\frac{x}{100}\right)^{y}-1\right\}}{\left\{\left(1+\frac{x}{100}\right)-1\right\}}$$

$$\therefore \quad a = \frac{(\text{TLC})\left(\frac{x}{100}\right)\left(\frac{100+x}{100}\right)^{y-1}}{\left(\frac{100+x}{100}\right)^{y}-1} \qquad (5.24)$$

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For lining to be economically feasible, the value of a should be less than the annual benefit B_t i.e., the ratio B_t/a should be greater than unity.

Types of Lining

Types of lining are generally classified according to the materials used for their construction. Concrete, rock masonry, brick masonry, bentonite-earth mixtures, natural clays of low permeability, and different mixtures of rubble, plastic, and asphaltic materials are the commonlyused materials for canal lining. The suitability of the lining material is decided by: (i) economy,
structural stability, (iii) durability, (iv) reparability, (v) impermeability, (vi) hydraulic efficiency, and (vii) resistance to erosion (15). The principal types of lining are as follows:

Concrete lining,
Shotcrete lining,
Precast concrete lining,
Lime concrete lining,
Stone masonry lining,
Brick lining,
Boulder lining,
Asphaltic lining, and
Earth lining.

Concrete Lining

Concrete lining is probably the best type of lining. It fulfils practically all the requirements of lining. It is durable, impervious, and requires least maintenance. The smooth surface of the concrete lining increases the conveyance of the channel. Properly constructed concrete lining can easily last about 40 years. Concrete linings are suitable for all sizes of channels and for both high and low velocities. The lining cost is, however, high and can be reduced by using mechanised methods.

The thickness of concrete depends on canal size, bank stability, amount of reinforcement, and climatic conditions. Small channels in warm climates require relatively thin linings.

Channel banks are kept at self-supporting slope (1.5H: 1V to 1.25H: 1V) so that the lining is not required to bear earth pressures and its thickness does not increase. Concrete linings are laid without form work and, hence, the workability of concrete should be good. Also, experiencedworkmen are required for laying concrete linings.

Reinforcement in concrete linings usually varies from 0.1 to 0.4% of the area in the longitudinal direction and 0.1 to 0.2% of the area in the transverse direction. The reinforcement in concrete linings prevents serious cracking of concrete to reduce leakage, and ties adjacent sections of the lining together to provide increased strength against settlement damage due to unstable subgrade soils or other factors. The reinforcement in concrete linings does not prevent the development of small shrinkage which tend to close when canals are operated and linings are watersoaked. The damage due to shrinkage and temperature changes is avoided or reduced by the use of special construction joints. Reinforced concrete linings may result in increased watertightness of the lining. However, well-constructed unreinforced concrete linings may be almost equally watertight.

The earlier practice of using reinforced concrete linings is now being replaced by the employment of well-constructed unreinforced concrete linings. However, reinforcement must be provided in: (a) large canals which are to be operated throughout the year, (b)

sections where the unreinforced lining may not be safe, and (c) canals in which flow velocities are likely to be very high.
Proper preparation of subgrade is essential for the success of the concrete lining which may, otherwise, develop cracks due to settlement. Natural earth is generally satisfactory for this purpose and, hence, subgrade preparation is the least for channels in excavation. Thorough compaction of subgrade for channels in filling is essential for avoiding cracks in lining due to settlement.
Some cracks usually develop in concrete linings. These can be sealed with asphaltic compounds. The lining may be damaged when flow in the canal is suddenly stopped and the surrounding water table is higher than the canal bed. This damage occurs in excavated channels and can be prevented by providing weep holes in the lining or installing drains with outlets in the canal section.
Values of minimum thickness of concrete lining based on canal capacity have been specified as given in Table 5.5.

Table 5.5 Thickness of concrete lining (14)

Canal capacity (m^3/s)	Thickness of M-150 concrete (cm)		Thickness of M-100 concrete (cm)	
	Controlled	Ordinary	Controlled	Ordinary
0 to less than 5	5.0	6.5	7.5	7.5
5 to less than 15	6.5	6.5	7.5	7.5
15 to less than 50	8.0	9.0	10.0	10.0
50 to less than 100	9.0	10.0	12.5	12.5
100 and above	10.0	10.0	12.5	15.0

Concrete linings have been used in the Nangal Hydel canal. Amaravathi project, the Krishnagiri Reservoir project, and several other projects. The use of concrete lining in India is, however, limited because of the low cost of water and high cost of lining. The Bureau of Indian Standards does not specify use of reinforcement for cement concrete lining.

Shotcrete Lining

Shotcrete lining is constructed by applying cement mortar pneumatically to the canal surface. Cement mortar does not contain coarse aggregates and, therefore, the proportion of cement is higher in shotcrete mix than in concrete lining. The shotcrete mix is forced under pressure through a nozzle of small diameter and, hence, the size of sand particles in the mix should not exceed 0.5 cm. Equipment needed for laying shotcrete lining is light, portable, and of smaller size compared to the equipment for concrete lining. The thickness of the shotcrete lining may vary from 2.5 to 7.5 cm. The preferred thickness is from 4 to 5 cm.
Shotcrete lining is suitable for: (a) lining small sections, (b) placing linings on irregular surfaces without any need to prepare the subgrade, (c) placing linings around curves or structures, and (d) repairing badly cracked and leaky old concrete linings.
Shotcrete linings are subject to cracking and may be reinforced or unreinforced. Earlier, shotcrete linings were usually reinforced. A larger thickness of shotcrete lining was preferred for the convenient placement of reinforcement. The reinforcement was in the form of wire mesh. In order to reduce costs, shotcrete linings are not reinforced these days, particularly on relatively small jobs.

Precast Concrete Lining

Precast concrete slabs, laid properly on carefully prepared subgrades and with the joints effectively sealed, constitute a serviceable type of lining. The precast slabs are about 5 to 8 cm thick with suitable width and length to suit channel dimensions and to result in weights which can be conveniently handled. Such slabs may or may not be reinforced. This type of lining is best suited for repair work as it can be placed rapidly without long interruptions in canal operation. The side slopes of the Tungabhadra project canals have been lined with precast concrete slabs.

Lime Concrete Lining

The use of this type of lining is limited to small and medium size irrigation channels with capacities of up to 200 m^3/s and in which the velocity of water does not exceed 2 m/s (16). The materials required for this type of lining are lime, sand, coarse aggregate, and water. The lime concrete mix should be such that it has a minimum compressive strength of about 5.00 kN/m^2 after 28 days of moist curing. Usually lime concrete is prepared with 1 : 1.5 : 3 of kankar lime : kankar grit or sand : kankar (or stone or brick ballast) aggregate. The thickness of the lining may vary from 10 to 15 cm for discharge ranges of up to 200 m^3/s. Lime concrete lining has been used in the Bikaner canal taking off from the left bank of the Sutlej.

Stone Masonry Lining

Stone masonry linings are laid on the canal surface with cement mortar or lime mortar. The thickness of the stone masonry is about 30 cm. The surface of the stone masonry may be smooth plastered to increase the hydraulic efficiency of the canal. Stone masonry linings are stable, durable, erosion-resistant, and very effective in reducing seepage losses. Such lining is very suitable where only unskilled labour is available and suitable quarried rock is available at low price. This lining has been used in the Tungabhadra project.

5.14.6.8. Brick Lining

Bricks are laid in layers of two with about 1.25 cm of 1 : 3 cement mortar sandwiched in between. Good quality bricks should be used and these should be soaked well in water before being laid on the moistened canal surface.

Brick lining is suitable when concrete is expensive and skilled labour is not available. Brick lining is favoured where conditions of low wages, absence of mechanisations, shortage of cement and inadequate means of transportation exist. Brick linings have been extensively used in north India. The Sarda power channel has been lined with bricks. The thickness of thebrick lining remains fixed even if the subgrade is uneven. Brick lining can be easily laid in rounded sections without form work. Rigid control in brick masonry is not necessary. Sometimesreinforced brick linings are also used.

Boulder Lining

Boulder lining of canals, if economically feasible, is useful for preventing erosion and where the ground water level is above the bed of the canal and there is a possibility of occurrence of damaging back pressures (17). The stones used for boulder linings should be sound, hard,durable, and capable of sustaining weathering and water action. Rounded or sub-angular river cobbles or blasted rock pieces with sufficient base area are recommended types of stones for boulder lining. Dimensions of stones and thickness of lining are as given in Table 5.6.

Table 5.6 Dimensions of stones and thickness of lining (17)

Canal capacity (m^3/s)	Thickness of lining (mm)	Average dimension along the longest axis	Minimum dimension at any

		(mm)	section (mm)
0 to less than 50	150	150	75
50 to less than 100	225	225	110
100 and above	300	300	150

Wherever required, a 15-cm thick layer of filter material is to be provided. For the laying of boulders, the subgrade (both bed and side slope) of the canal is divided into compartments by stone masonry or concrete ribs. These compartments will not have dimensions more than 15 m along and across the centre line of the canal.

Asphaltic Lining

The material used for asphaltic lining is asphalt-based combination of cement and sand mixed in hot condition. The most commonly used asphaltic linings are: (a) asphaltic concrete, and (b) buried asphaltic membrane. Asphaltic linings are relatively cheaper, flexible, and can be rapidly laid in any time of year. Because of their flexibility, minor movements of the subgrade are not of serious concern. However, asphaltic linings have short life and are unable to permit high velocity of flow. They have low resistance to weed growth and, hence, it is advisable to sterilise the subgrade to prevent weed growth.

Asphaltic concrete is a mixture of asphalt cement, sand, and gravel mixed at a temperature of about 110°C and is placed either manually or with laying equipment. Experienced and trained workmen are required for the purpose. The lining is compacted with heavy iron plates while it is hot.

A properly constructed asphaltic concrete lining is the best of all asphaltic linings. Asphaltic concrete lining is smooth, flexible, and erosion-resistant. Since asphaltic concrete lining becomes distorted at higher temperatures, it is unsuitable for warmer climatic regions. An asphaltic concrete lining is preferred to a concrete lining in situations where the aggregate is likely to react with the alkali constituents of Portland cement.

Buried asphaltic membrane can be of two types:

Hot-sprayed asphaltic membrane, and

Pre-fabricated asphaltic membrane.

A hot-sprayed asphaltic membrane is constructed by spraying hot asphalt on the subgrade to result in a layer about 6 mm thick. This layer, after cooling, is covered with a layer of earth material about 30 cm thick. The asphalt temperature is around 200°C and the spraying pressure about 3×10^5 N/m^2. For this type of lining, the channel has to be over-excavated. The lining is flexible and easily adopts to the subgrade surface. Skilled workmen are required for the construction of this type of lining.

Pre-fabricated asphaltic membrane is prepared by coating rolls of heavy paper with a 5 mm layer of asphalt or 3 mm of glass fibre-reinforced asphalt. These rolls of pre-fabricated asphaltic membrane are laid on the subgrade and then covered with earth material. These linings can be constructed by commonly available labour.

Materials used for covering the asphaltic membrane determine the permissible velocities which are generally lower than the velocities in unlined canals. Maintenance cost of such linings is high. Cleaning operations should be carried out carefully so as not to damage the membrane.

Earth Linings

Different types of earth linings have been used in irrigation canals. They are inexpensive but require high maintenance expenditure. The main types of earth linings are: (a) stabilised earth linings, (b) loose earth blankets, (c) compacted earth linings, (d) buried bentonite membranes, and (e) soil-cement linings.

Stabilised earth linings: Stabilised earth linings are constructed by stabilizing the subgrade. This can be done either physically or chemically. Physically stabilised linings are constructed by adding corrective materials (such as clay for granular subgrade) to the subgrade, mixing, and then compacting. If corrective materials are not required, the subgrade can be stabilised by scarifying, adding moisture, and then compacting. Chemically stabilised linings use chemicals which may tighten the soil. Such use of chemicals, however, has not developed much.

Loose earth blankets: This type of lining is constructed by dumping fine-grained soils, such as clay, on the subgrade and spreading it so as to form a layer 15 to 30 cm thick. Such linings reduce seepage only temporarily and are soon removed by erosion unless covered with gravel. Better results can be obtained by saturating the clay and then pugging it before dumping on the subgrade. The layer of pugged clay is protected by a cover of about 30 cm silt. This type of lining requires flatter side slopes.

Compacted earth linings: These linings are constructed by placing graded soils on the subgrade and then compacting it. The graded soil should contain about 15% of clay. The compacted earth linings may be either thin-compacted or thick-compacted. In thin-compacted linings, the layer thickness of about 15 to 30 cm along the entire perimeter is used. Thick- compacted linings have a layer about 60 cm thick on the channel bed and 90 cm thick on the sides. If properly constructed, both types are reasonably satisfactory. However, the thick linings are generally preferred.

Compacted-earth linings are feasible when excavated materials are suitable, or when suitable materials are available nearby. Compaction operations along the side slopes are more difficult (particularly in thin-compacted linings) than along the channel bed. The lining material should be tested in the laboratory for density, permeability, and optimum moisture contents.The material must be compacted in the field so as to obtain the desired characteristics.

Buried Bentonite Membranes: Pure bentonite is a hydrous silicate of alumina. Natural deposits of bentonite are special types of clay soil which swell considerably when wetted. The impurities of these soils affect the swelling and, hence, the suitability of these as canal lining material. Buried bentonite linings are constructed by spreading soil-bentonite mixtures over the subgrade and covering it with about 15 to 30 cm of gravel or compacted earth. Sandy soil mixed with about 5 to 25 per cent of fine-grained bentonite and compacted to a thickness of 5 to 7.5 cm results in a membrane which is reasonably tough and suitable for lining.

Soil-cement Linings: These linings are constructed using cement (15 to 20 per cent by volume) and sandy soil (not containing more than about 35 per cent of silt and clay particles). Cement and sandy soil can be mixed in place and compacted at the optimum moisture content. This method of construction is termed the dry-mixed soil-cement method. Alternatively, soil-cement lining can be constructed by machine mixing the cement and soil with water and placing it on the subgrade in a suitable manner. This method is called the plastic soil-cement method and is preferable. In both these methods, the lining should be kept moist for about seven days to permit adequate curing.

The construction cost of soil-cement linings is relatively high. But these resist weed growth and erosion and also permit velocities slightly higher than those permitted by unlined earth channels. The use of soil-cement linings for irrigation canals is restricted to small irrigation canals with capacities of up to 10 m^3/s and in which the velocity of water does not exceed 1 m/s (18).

Failure of Lining

The main causes of failure of lining are the water pressure that builds up behind the lining material due to high water table, saturation of the embankment by canal water, sudden lowering of water levels in the channel, and saturation of the embankment sustained by continuous rainfall. The embankment of a relatively pervious soil does not need drainage measures behind the lining. In all situations requiring drainage measures to relieve pore pressure behind the lining, a series of longitudinal and transverse drains satisfying filter criteria are provided. A typical arrangement of longitudinal filter drain is as shown in Fig. 5.24.

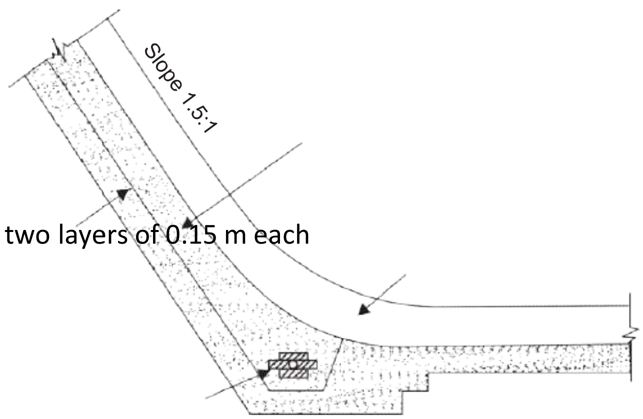


Fig. 5.24 Longitudinal filter drain

The growth of weeds on canal banks and other aquatic plant in channels may not result in failure of the lining but would affect the conveyance of channels which may be lined or unlined. Weeds and aquatic plants consume water for their growth and thus the consumptive use of irrigation water increases. Weed growth increases channel roughness and, hence, reduces the flow velocity thereby increasing evaporation losses. The cleaning of channels having excessive weed growth is, therefore, a vital maintenance problem. Cleaning operations can be carried out manually or by mechanical devices, such as used in dragline excavation and tractor- drawn cranes. Commonly used methods are pasturing, mowing, burning, and applying chemical weed killers.

DRAINAGE OF IRRIGATED LANDS

Drainage is defined as the removal of excess water and salts from adequately irrigated agricultural lands. The deep percolation losses from properly irrigated lands and seepage from reservoirs, canals, and watercourses make drainage necessary to maintain soil productivity. Irrigation and drainage are complementary to each other. In humid areas, drainage attains much greater importance than in arid regions. Irrigated lands require adequate drainage to remain capable of producing crops. The adequate drainage of fertile lands requires the lowering of a shallow water table, and this forms the first and basic step in the reclamation of waterlogged, saline, and alkali soils. The drainage of farm lands: (i) improves soil structure and increases the soil productivity, (ii) facilitates early ploughing and planting, (iii) increases the depth of root zone thereby increasing the available soil moisture and plant food, (iv) increases soil ventilation, (v) increases water infiltration into the ground thereby decreasing soil erosion on the surface, (vi) creates favourable conditions for growth of soil bacteria, (vii) leaches excess salts from soil, (viii) maintains favourable soil temperature, and (ix) improves sanitary and health conditions for the residents of the area.

The water table can be lowered by eliminating or controlling sources of excess water. An improvement in the natural drainage system and the provision of an artificial drainage system are of considerable help in the lowering of the water table. A natural drainage system can be properly maintained at low costs and is a feasible method of protecting irrigated lands from excessive percolation. Artificial drainage also aims at lowering the water table and is accomplished by any of the following methods:

Open ditch drains

Subsurface drains

Drainage wells

Open ditch drains (or open drains) are suitable and very often economical for surface and subsurface drainage. They permit easy entry of surface flow into the drains.

Open drains are used to convey excess water to distant outlets. These accelerate the removal of storm water and thus reduce the detention time thereby decreasing the percolation of water into the ground. Open drains can be either shallow surface drains or deep open drains. Shallow surface drains do not affect subsurface drainage. Deep open drains act as outlet drains for a closed drain system and collect surface drainage too.

The alignment of open drains follows the paths of natural drainage and low contours. The drains are not aligned across a pond or marshy land. Every drain has an outlet the elevation of which decides the bed and water surface elevations of the drain at maximum flow. The longitudinal slope of drain should be as large as possible and is decided on the basis of non-scouring velocities. The bed slope ranges from 0.0005 to 0.0015. Depths of about 1.5 to 3.5 m are generally adopted for open drains. The side slopes depend largely on the type of embankment soil and may vary from 1/2 H : 1V (in very stiff and compact clays) to 3H : 1V (in loose sandy formations).

The open drains should be designed to carry part of storm runoff also. The cross-section of open drain is decided using the general principles of channel design. The channel will be in cutting and the height of banks will be small. If the drain has to receive both seepage and storm water, it may be desirable to have a small drain in the bed of a large open drain. This will keep the bed of the drain dry for most of the year and maintenance problems will be considerably less. Only the central deeper section will require maintenance.

Open drains have the advantages of: (a) low initial cost, (b) simple construction, and (c) large capacity to handle surface runoff caused by precipitation. However, there are disadvantages too. Besides the cost of land which the open drains occupy and the need of constructing bridges across them, open drains cause: (a) difficulty in farming operations, and
(b) constant maintenance problems resulting from silt accumulation due to rapid weed growth in them.

Flow of clear water at low velocities permits considerable weed growth on the channel surface. The open drains have, therefore, to be cleaned frequently. In addition to manual cleaning, chemical weed killers are also used. But, at times the drain water is being used for cattles and the weed poison may be harmful to the cattles. Aquatic life is also adversely affected by the chemical weed killers.

Subsurface drainage (or underdrainage) involves the creation of permanent drainage system consisting of buried pipes (or channels) which remain out of sight and, therefore, do not interfere with the farming operations. The buried drainage system can remove excess water without occupying the land area. Therefore, there is no loss of farming area. Besides, there is no weed growth and no accumulation of rubbish and, therefore, the underdrainage system can remain effective for long periods with little or no need for maintenance. In some situations, however, siltation and blockage may require costly and troublesome maintenance or even complete replacement.

The materials of the buried pipes include clay pipes and concrete pipes in short lengths (permitting water entry at the joints) or long perforated and flexible plastic pipes. In addition, blankets of gravel laid in the soil, fibrous wood materials buried in the soil or such materialswhich can be covered by the soil and which will remain porous for long time are used for theconstruction of underdrainage system. If such drains are to be placed in impervious soil, thedrains should be surrounded by a filter of coarser material to increase the permeability and prevent migration of soil particles and blocking of drains.

Mole drains are also included as subsurface drains. The mole drains are unlined and unprotected channels of circular cross-section constructed in the subsoil at a depth of about
0.70 m by pulling a mole plough through the soil without digging a trench, Fig. 5.25. The mole plough is a cylindrical metal object (about 300 to 650 mm long and 50 to 80 mm in diameter) with one of its ends bullet-shaped. The mole is attached to a horizontal beam through a thin blade as shown in Fig. 5.26. A short cylindrical metal core or sphere is attached to the rear of the mole by means of a chain. This expander helps in giving a smooth finish to the channel surface. The basic purpose of all these subsurface drains is to collect the water that flows in the subsurface region and to carry this water into an outlet

channel or conveyance structure. The outlets can be either gravity outlets or pump outlets. The depth and spacing of the subsurface drains (and also deep open drains) are usually decided using Hooghoudt's equation described in the following.

Slot left by mole blade

Mole channel

Fig. 5.25. Mole drain

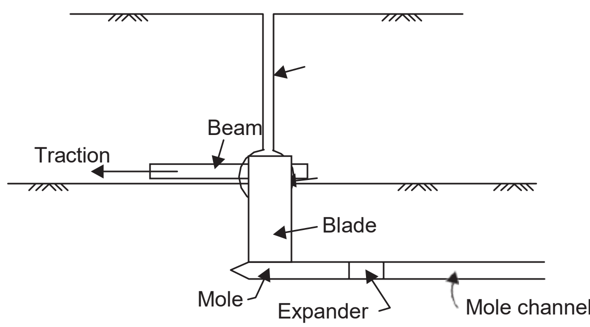


Fig. 5.26 Mole plough

INADEQUACIES OF CANAL IRRIGATION MANAGEMENT

From the point of view of performance, the management of the canal irrigation systems in India is far from satisfactory. The major inadequacies are as follows.

1. Insufficient planning and preparation at the stage of execution of the project which results in longer construction time and escalated project cost,
2. Involvement of more than one ministry/department and poor coordination among them,
3. Non-responsive, authoritarian, and poor administration resulting in increased mal- practices,
4. Lack of interaction between engineering and agricultural experts,
5. Lag between creation of potential and its utilisation,
6. Improper assessment of personnel, equipment, and other facilities for proper opera- tion and maintenance of reservoirs and canal systems resulting in erratic (unreli- able and insufficient) supplies and inequitable distribution of available water,
7. Higher conveyance losses,
8. Absence of conjunctive use of ground water and surface water,
9. Insufficient drainage, excessive seepage, and waterlogging,
10. Poor on-farm management,
11. Absence of farmer's participation in the management,
12. Lack of communication facilities in the command area,
13. Poor extension services – lack of pilot projects, demonstration farms, etc.,
14. Problems related to land settlement and rehabilitation of displaced persons, and
15. Recovery of the project cost

OBJECTIVES AND CRITERIA OF GOOD CANAL IRRIGATIONMANAGEMENT

There are several conventional measures to improve the performance of canal irrigation systems. Some of these measures are lining of canals and field channels, on-farm development, farmers' organisation, warabandi system of water distribution, charging farmers volumetrically for water, and educating farmers in water use management. However, before seeking a solution to improve the irrigation management, it is worthwhile to consider the objectives of irrigation and the criteria for judging the performance of an irrigation project.

The effects or impacts of irrigation can be best phrased as ''optimising human well-being'' (1). The term ''well-being'' includes food security, incomes, nutritional status, health, education, amenity, social harmony, and self-respect.

The criteria for judging the performance of canal irrigation systems can be vastly different for different groups of people depending upon their concerns (Table 6.5). However, the most common criteria generally accepted for judging the performance of an irrigation system are productivity, equity, and stability which together contribute to the objective of well-being.

Table 6.5 Criteria of good irrigation system performance (1)

Type of person	Possible first criterion of good system performance
Landless Labourer	Increased labour demand, days of working, and wages
Farmer	Delivery to his or her farm of an adequate, convenient, predictable, and timely water supply for preferred farming practices
Irrigation engineer	Efficient delivery of water from headworks to outlet
Agricultural engineer	Efficient delivery and field application of irrigation water from theoutlet to the root zone of the crop
Agronomist	Creation and maintenance of the optimum moisture regime and plant growth and, in particular, maximising production of that part of theplant which is the harvestable product
Agricultural economist	High and stable farm production and incomes
General economist	A high internal rate of return
Political economist	Equitable distribution of benefits, especially to disadvantaged groups
Sociologist	Participation of irrigators in management

Productivity

Productivity is defined as the ratio of output and input. The output can be water delivered, area irrigated, yield, or income, and the input can be water in the root zone, at the farm gate, at the outlet or at upstream points in the system including the point of diversion or storage (10). Fig. 6.1 shows typical points of input and output measurements for different professionals (1). Improved water supply influences the adoption of high-yielding agricultural practices byfarmers which justifies the productivity criterion of performance.

Equity

Equity in canal irrigation systems implies equality, fairness, and even-handed dealing in matters of allocation and appropriation of irrigation water (1). There can be several ways to decide the equality of supplies to different farmers. Two of them, practised throughout the world, are the methods of prior appropriation and of proportionate equality. In the method of prior appropriation, whoever first exploits a resource establishes a right to continue to do so. Thus, head reach farmers and early comers to an irrigation project establish their right over irrigation water, even if it means less water or no water to tail-end farmers or late comers. In the second method of proportionate equality, the supply of water is in proportion to the size of the land- holding as in the warabandi system of north-west India. In India, on most canal irrigation systems, water distribution is far from meeting even the criterion of water proportional to land and, hence, both these methods have been criticised for their inequity (1). Attempts to improve equity are usually limited to achieving the supply of water in proportion to the size of the land.

Stability

Stability of productivity as well as equity are important. Stability can either be short-term stability or long-term sustainability. The short-term or interseasonal stability refers to the variations in productivity and equity between irrigation seasons, and is a function of climate, water supply, storage and control, system management, and other factors such as pests, diseases, and availability of labour and other inputs. It can be measured by comparing performance between seasons.

The long-term sustainability has been described as ''environmental stability'' and ''durability'' and refers to the prevention or minimising of adverse physical changes such as waterlogging, leaching of nutrients from soils, salinity, erosion, silting, the 'mining' of ground water, and infestations with weeds. Sustainability can be monitored by measuring ground water levels, salinity, erosion, or silting through inspecting works, and by measuring long-term trends in productivity and equity.

Well-being is a broad objective achieved through productivity, equity, and stability. There are several aspects of well-being which must be borne in mind at all stages of the project with both positive and negative effects. These aspects include health, nutrition, amenity (especially water for washing and bathing, raising ground water for domestic purposes, and so on), and psychological factors such as freedom from domination, feeling of participation, etc.

METHODS FOR IMPROVING CANAL IRRIGATION MANAGEMENT

Irrigation management is an interdisciplinary system process with a built-in learning mechanism to improve system performance by adjusting physical, technological, and institutional inputs to achieve the desired levels of output. Canal irrigation is a complex process involving physical, bio-economic, and human activities which are interrelated and vary widely over space and time. As such, canal irrigation management demands special methods. Every management problem requires to be analysed in detail and then solved accordingly. Nevertheless, there are some aspects which, if considered properly at different stages, can help significantly in the improvement of canal irrigation. These aspects have been briefly dealt with in the following.

Cropping Patern

Cropping pattern is described in terms of the area under various crops at different periods of a year. An optimum cropping pattern for an area can ideally be determined by using systems analysis. If the local preferences and requirements of the area are included in the analysis, and the necessary inputs are made available, the farmers will adopt the cropping pattern arrived at using systems analysis. One such analysis carried out for the Gomti-Kalyani doab under Sarda Sahayak command has recommended a cropping pattern which is expected to increase the net annual benefit to Rs. 23.67 crore from the existing benefit of Rs. 11.57 crore.

Conjunctive Use

Often in the past, the development of water resources has taken place in such a manner as if surface water and ground water were two separate sources. Successful management of water resources requires adding to the two-dimensional development of surface water the third dimension of depth to include ground water.

Conjunctive use means that water lifted from below the ground is used in conjunction with canal waters. It results in the coordinated, combined, and creative exploitation of ground water and surface water so as to minimise the dislocation caused by nature's inconsistent rainfall pattern (16). Conjunctive use implies use of surface water (from either reservoir storage or diversion works) during periods of above normal precipitation for irrigation and other activities to the extent possible and letting the balance reach the ground water storage (through artificial recharge) which would be utilised for supplementing surface water supplies during years of subnormal precipitation. Such coordinated use of surface and ground waters results in increased amount of available water, smaller surface distribution system, smaller drainage system, reduced canal linings, greater flood control, and smaller evaporation losses. There are, however, some disadvantages too in resorting to conjunctive use. These are lesser hydroelectric power, greater power requirement, need for artificial recharge, and danger of land subsidence. The parameters related to conjunctive use, such as cropping pattern, canal capacities, capacities and spacing of wells, drainage requirements, optimum ground water level, etc. are best determined by systems analysis to derive maximum benefits.

While studying the available water resources and the original plans of Mahi-Kadana project of Gujarat which did not include extensive use of ground water, Sarma et al. (17) proposed the conjunctive use of water. Their calculations, based on a culturable command area of 213,000 ha, indicated that the intensity of irrigation could be raised from 55 per cent (achieved in 1980–81) to 180 per cent through conjunctive use. Besides, the rise in the groundwater table would also be arrested.

Channel Capacity

The discharge capacity of the channel system should be decided on the concept of evapotranspiration rather than the 'kor' period.

Canal Lining

Lining of canals is a means to reduce the seepage losses from canals. In one typical case, the benefit-cost ratios of lining distributaries only, distributaries and watercourses, and field channels only were found (18) to be, respectively, 0.33, 0.608 and 2.303. As such, the lining of field channels is the most beneficial. Besides, it involves no dislocation in the operation of an existing system. In order to prevent damage to lining, the slope of a lined channel is reduced. This reduces the sediment carrying capacity of an existing channel which is being lined. Therefore, measures for sediment exclusion are to be considered whenever an existing canal is being lined. Alternative to the lining of canals is the conjunctive use of surface and ground water which should be opted for after comparing the unit cost of water saved by lining with the unit cost of pumped water. For a representative case, Chawla (16) has worked out the cost of water saved by lining as Rs. 25 for every 100 m^3 of water against Rs. 7 to Rs. 12 for the same amount of pumped water.

Regulators and Escapes

For ensuring proper distribution of irrigation water according to the adopted management policy, a suitable number of canal regulators and canal escapes must be provided on the channel network in general and on main canals and branches in particular. Canal escapes are needed for the safety as well as for regulating canal supplies in areas which have received excess rainfall.

Canal Outlets

Another important aspect of designing canal irrigation system is the selection of suitable type of outlet which is crucial in controlling the distribution of water and providing a link between the administration and the farmer. From the considerations of equitable distribution of water, a regulated outlet would be an ideal choice provided that it can be operated efficiently and honestly. Unfortunately, the present socio-political conditions prevailing in the country, however, do not permit such operation (8). As such, for the present, regulated outlets are ruled out. The subproportional semi-modules would be the right choice for locations just upstream of falls and regulators and also on branch canals (8). Other outlets may be of non-modular type.

Main System Management

Operational management of the main system refers to management aspects of the future allocation, scheduling, delivery of water on main systems down to and including outlets, and the disposal of water in drains below chaks (i.e., the irrigated fields) (1). It includes planning, decision making, the operation of controls, and communications both upwards to managers and downwards to groups of farmers so that equitable supplies can be ensured throughout the command area (1). Main system management (MSM) is capable of reducing gross inequities of water supply to tail-end farmers and increasing the farm yield from the command area. A suitable type of MSM can induce the farmers for active involvement in on-farm development and maintenance and adoption of high-yielding practices. The importance of MSM is more in reservoir-based irrigation systems than in the systems based on diversion schemes because of the available options of storage and release.

According to the present system of operation, the main canal and branches run continuously with either full or reduced supply depending on the availability of water. The distributaries and minors, however, run intermittently in accordance with the keenness of demand as assessed by the concerned field staff. Such assessment tends to be subjective as well as approximate. A more rational method for the running schedule of distributaries and minors can, alternatively, be worked out as follows (8):

Obtain the cropping pattern, preferably an optimum one, for irrigation during the ensuing season,

Estimate weekly evaportranspiration and corresponding effective rainfall based on past records,

Determine the irrigation demand,

Decide upon the amount of canal water and ground water to meet the irrigation demand such that the desired intensity of irrigation on the optimum cropping pat- tern can be obtained along with a stable water table, and

Prepare a roster of regulation of distributaries and minors and notify the concerned farmers well in advance to enable them to plan their sowing and irrigation pro- grammes accordingly.

Provision for departure from the notified roster should be available. Such departures should be made only if there is variation in rainfall pattern.

Night Irrigation

Another important issue in the operation of the main system is related to night irrigation. In most of the canal irrigation projects, the canal water continues to flow at nights as well and is either badly used or wasted. Darkness, cold, fear, normal working hours and desire for sleep discourage the irrigation staff, farmers, and labourers to work at night. In India, about 47 per cent of the 24 hours day is the time of darkness (between sunset to sunrise with an allowance of 20 minute twilight period each after the sunset and before the sunrise). According to Chambers' approximate estimate, about 40 per cent of the canal irrigation water on medium and major systems (i.e., with commands over 2000 ha) is either applied in night irrigation or sent into drains at night (1). He further estimates that, except in north-west India where warabandi is practised at night, 25 per cent of the canal irrigation water is wasted at night and much of the 15 per cent which is applied at night is used inefficiently. In view of the magnitude of these losses, night irrigation is too important to remain neglected any more. Farmers usually dislike night irrigation for the following reasons (1):

Loss of sleep and disruption in the normal sleeping duration,

Discomfort due to cold night and difficulty in moving around in sticky soils and mud, Danger and fear of snakes, scorpions, accidents, violence including murder, and other problems related to law and order,

Inefficient application of water due to darkness, and

Higher costs due to higher night wages, non-availability of family labour, especiallywomen, old people and those very young to work at night, and need of firewood, beverages and lighting.

In addition, field conditions, type of crop, and its stage of growth may add to the difficultiesof irrigating at night. Sloping lands with difficult soils and standing crops are difficult to be irrigated at night. Paddy, trees, and crops which are low and wide apart or are in early stagesof growth can be irrigated with relative ease. High and dense crops, and crops in the later stages of growth are relatively difficult to be irrigated at night. The difficulties in night irrigation may influence a farmer’s choice of crop if he has to rely on only water received at night.

However, sometimes night irrigation is preferred by farmers due to the following reasons (1):

In warmer regions, farmers find it more comfortable to irrigate at night.

Part-time farmers having other work during the day would prefer night irrigation.

Tail-end farmers may get relatively more adequate and reliable supply during night. In addition to the above characteristics of night irrigation, the following features must also be noted (1):

It is during the night that illicit appropriation of water (breaching of bunds, remov- ing checks, blocking streams, opening pipes, pumping out of channels etc. to secure water for an individual or a group) takes place.

During nights, evaporation losses are much less. This saving is, however, more than offset by wastage.

Most physical damages such as erosion in steep minors, watercourses and field chan- nels due to relatively larger flows on account of lesser withdrawals upstream, and aggravation of waterlogging and associated soil salinity and alkalinity problems seem to occur at night.

It is, therefore, obvious that night irrigation presents many problems. This situation can be improved either by reducing irrigation at night or improving it. Reduction in night irrigation can be either with or without saving water for subsequent use. Measures to reducenight irrigation with water saving involve storage of water which can be achieved in five different ways – in main reservoirs, in canals, in intermediate storage reservoirs, on-farm storage, andin ground water storage. On the other hand, three obvious measures to reduce night irrigation without saving water are: (i) stopping river diversion flows, (ii) redistributing day water so that even tail-enders get adequate supplies, and (iii) passing water to escapes and drains. For obvious reasons, measures with water saving are more desirable but much needs to be done to make them practicable. Night irrigation can be improved in the following ways (1):

Making flows predictable and manageable. For this purpose, the warabandi system has obvious advantages. Each farmer knows the time of his turn and does not needto recruit labour to capture and guard his supply.

Improving convenience and efficiency by measures such as (a) good lighting, (b) or- ganisation of groups of farmers for night irrigation for mutual benefits of shared labour and common protection from water raiders, etc., (c) installing and maintain ing structures, channels, fields and water application methods requiring minimumobservation and adjustment, (d) shaping fields for easier water application, and (e) providing ways and means for easy movement from residences to the fields and withinthe fields.

Choosing easy crops such as paddy and trees and crops which are low and wideapart.
Zoning for night flows which means using day flows for more difficult crops and soils and night flows for relatively easier crops and soils.
Phasing night irrigation for shorter, warmer, and moon-lit nights would, obviously, be more convenient for farmers.
It should, however, be noted that night irrigation had not received any attention till recently. Only a beginning has been made and lot more needs to be done to mitigate the problems related to night irrigation.

Water Delivery System

Water delivery systems can be of three types:
Demand-based,
Continuous, and
Rotational (also known as warabandi).

Of these three, the warabandi system (Sec. 5.11) seems to be the most feasible and offers many advantages. It has been defined (19) as a system of equitable water distribution by turns according to a predetermined schedule specifying the day, time, and duration of supply to each irrigator in proportion to land holdings in the outlet command. Both the Irrigation Commission of 1972 and the National Commission on Agriculture of 1976 saw warabandi, with fixed times but taking water throughout the 24 hours, as a means of tackling waste of water at night (1). The procedure for estimating the entitlement of a farmer varies in details. For example, in UP and north-western states, the entitlement is based only on the area of the farm whereas in Maharashtra and other southern states, the farmer has to seek approval for irrigation of specific areas for specific crops (8). Some studies have indicated that the warabandi system, although efficient and acceptable to the farmers, does not result in equitable distribution primarily due to the losses in the watercourses. These losses may cause about 25 to 40 per cent reduction in the share of water of the farmer in the tail-end reach. While determining the schedule, some weightage should be given to the time allotment for tail-enders to compensate for the losses.

Another promising system of water delivery is through water cooperatives which purchase water in bulk and then distribute it among their member farmers. One example of this distribution is the Mohini Water Cooperative Society near Surat which is considered the firstsuccessful irrigation cooperative in Gujarat (20).

Irrigation Scheduling

For efficient management of an irrigation system, it is necessary that the water be supplied to the plants when they need it and in quantities actually required by the plants. This necessity leads one to irrigation scheduling which means estimating the starting time, stopping time, and the quantity of water for different cycles of irrigation during the crop period. Irrigation scheduling can be determined by using one of three approaches, viz., (i) the soil-moisture depletion approach, (ii) the climatological approach using evaportranspiration and effective rainfall data, and (iii) the farmer's existing schedule approach. A study on optimal irrigation scheduling for wheat crop of Udaipur region (21) has indicated that the soil-moisture depletion approach results in maximum water use efficiency.

Irrigation Methods

Most of the surface irrigation methods (Sec. 3.10) yield reasonably high field application efficiency provided the land has been prepared properly and due care has been taken during irrigation. However, the sprinkler method of irrigation and the drip irrigation method seem to be more promising than others in most of the conditions. The methods, however, require much higher initial investment, energy for generating pressure, and silt-free water. If the cost of land preparation and the percolation losses are high, sprinkler irrigation may result in considerable saving of money as well as water. The average cost of a sprinkler irrigation system may be approximately Rs. 15,000 per hectare and can possibly be recovered in about 2 years' time (22). The drip irrigation method is highly efficient and better suited for fruit crops, vegetables, and cash crops like sugarcane, cotton, groundnut, etc.. The cost of drip irrigation system may be around Rs. 30,000 per hectare. Table 6.6 compares different irrigation methods and clearly shows the superiority of sprinkler and drip irrigation systems over the surface methods (i.e., flooding, check, basin, border strip, and furrow methods).

Table 6.6 Comparison of suitability of irrigation methods

Site factor	Surface Method	Sprinkler Method	Drip Method
Soil Topography Crops Water supply Water quality Efficiency Labour requirement Capital requirement Energy requirement Management skill Weather	Uniform with moderate to low infiltration Level to moderate slopes All Large streams All but very salty 50- 80% High Low to high Low Moderate All	All Level to rolling Tall crops limit type of system Small stream nearly continuous Salty water may harm plants 70-80% Low to moderate Moderate to high Moderate to high Moderate Poor in windy conditions	All All High Value Small stream continuous and clean All 80-90% Low to high High Low to moderate High All

In India, sprinkler as well as drip irrigation have considerable scope because of the need to save water and extend irrigation facility to as large a cropped area as possible for producing food for the growing population. Sprinklers must find useful application in undulating sandy terrains of Rajasthan, Gujarat, Haryana, and Punjab. Drip irrigation is ideal for fruit orchards vegetable crops, and some cash crops. Water saved due to the introduction of these two methods in favourable regions may be enough to increase the irrigation potential for additional 5 million hectares (8).

Use of Waste Water

By AD 2000, the requirement of domestic water needs and thermal power plant needs will be around 3 Mha.m most of which will be used for non-agricultural purposes (8). It would, obviously, be very beneficial even if half of this used water is suitably treated and used for irrigation. Such measures can provide additional irrigation potential for about 1.5 to 2 Mha of cropped land (8).

Conservation of Water on the Field

Rice fields have to be kept flooded for a sufficiently long time, and this results in large percolation losses from 50 to 80 per cent depending upon the type of soil (22). Therefore, rice cultivation should normally be restricted to soils of relatively low permeability. The percolation losses can also be reduced by puddling the soil using improved puddlers and the saving of water can be between 16 and 26 per cent depending upon the type of soil and puddlers used (22).

Waterlogging

Waterlogging (Sec. 5.14) results in lowered yields, loss of lands for useful activities, and health hazards. To eliminate or control waterlogging one or more of the following, remedial measures have usually been used (1, 8):

Reducing inflow to the ground through lining of canals,
Removing ground water through pumping,
Removing surface and ground waters through drainage,
Educating farmers in water management, and
conjunctive use.

Of all these methods, the conjunctive use of surface and ground water is the most cost-effective means of fighting waterlogging in canal-irrigated lands. This has already been effectively tried in parts of western UP, Haryana, and Punjab (8). Waterlogging can also be reduced by supplying less water during nights (as was done on the head reach of the Morna system in Maharashtra), cutting off water supplies during rains, rotating supplies in distributaries and minors instead of continuous supply, shortening irrigation periods, and zoning for crop type (1).

Soil Reclamation

Saline soils are found in the states of Madhya Pradesh, Rajasthan, Maharashtra, Karnataka, Andhra Pradesh, West Bengal, Tamil Nadu, and Gujarat. Alkaline soils are found in the Indo- Gangetic plains of Punjab, Haryana, UP, and parts of Bihar and Rajasthan. Because of their adverse effects on agricultural production as well as magnitude, saline and alkaline soils need to be reclaimed on a high priority basis and in a planned manner by the joint efforts of agricultural chemists, agronomists, agricultural experts, and irrigation engineers. The role of an irrigation engineer is important in lowering the water table if it is high and also providing irrigation water of good quality for leaching out the salts.

For reclamation works, one needs to know the following before taking any remedial measure (8):

Characteristics of the soil and the salts present in it,
Availability and quality of irrigation water,
Level of ground water table, and Crops which can be grown under given conditions.

The main components of soil reclamation works are as follows (8):

Isolation of land areas according to their categorisation and levelling and bunding of the affected land as per the category.
Provision of drainage (surface or subsurface or vertical) network to remove leaching water and to keep the water table to a safer level.
Breaking up of impervious subsoil layer in alkali soils by deep ploughing.
Adding suitable chemicals (such as gypsum, sulphur, etc.) depending upon the re- sults of chemical tests of the affected soil.
After application of the chemicals, the leaching is carried out by four to five applications of good quality water up to depth of about 60 cm.
If the reclaimed soil is not suitable for foodgrain production, it can be used to grow certain species of trees such as safeda (hybrid eucalyptus), vilayati babul (prospis jiliflora), and indigenous babul (acacia milotica). If the soil has been reclaimed to the extent that it canbe used for foodgrain production, it is usual to grow paddy as the first crop in Kharif and barley or wheat in the next Rabi and dhaincha in summer as a green manure.

Water Charges and Pricing of the Agricultural Output

In India, canal water charges are generally fixed on the basis of area served and the crop irrespective of the volume of water supplied. Such a method of charging, obviously, leads to inefficient use of water. Due to the large number of small farmers, however, it is impractical to meter the supplies. An alternative, in the form of cooperative societies receiving the suppliesin bulk and managing internal distribution and collection of charges from its members, may possibly result in efficient use of water. One such society (the Mohini Water Cooperative Society near Surat in Gujarat) has been operating successfully in a sugarcane belt of Gujarat.

Command Area Development Programme

Eeffective management under these conditions is extremely difficult, if not impossible''. All these suggest that the CAD programme and the main system management must work in unison to obtain maximum benefits.

Farmer's Participation

The farmer forms the target group for all irrigation management and, hence, is a good sourceof first hand information right from the investigation to the operational stage of any irrigation system. It is, therefore, recognised that the participation of the farmer in all stages of the irrigation system would be very beneficial. Yoganarasimhan (28) has suggested farmer's participation in the following three phases:

Phase 1 – Initiation

Publicising the proposal, without restriction, as a promotional measure to gainpopular support.
Extending invitations for information meeting.
Scheduling public hearing.
Warranting the critical assessment of objections raised as well as of individualalternatives of supplementary proposals.
Facilitating discussions and debates on proposal components and details.
Arranging for secret ballots to be cast in preparation of final decision.
Facilitating the nomination and election of farmer's representatives to join working groups.
Drafting the statutes of organisation which may only be adopted by majority vote through secret ballot.
Imparting courage, inspiration, and resolution to farmers.
Creating an atmosphere of trust and confidence in the feasibility of local participation.

Phase 2 – Continuation

Establishing formal organisations of farmers.

Recognising the roles of organisation.

Forming joint committees charged with proposing and reviewing.

Conducting training courses.

Inviting contributions from farmers in the process of implementation, management, operation and maintenance, and in situations where repairs are required.

Creating an atmosphere of collaboration and cooperation.

Strengthening the task-oriented leadership based on majority group consensus and support.

Phase 3 – Perpetuation

Assisting farmers to maintain their level of participation and increase its intensity.

Relating their duties to improve their socio-economic status.

Monitoring the performance of programmes to generate feedback into assistance and instruction.

Inducing farmers to actively participate in the management through unreserved recognition of their achievements.

Irrigation Manager

Another important factor is the set of people who manage canal irrigation. Any improvement in the existing canal irrigation management, viz., scheduling, reducing losses at night, etc. can only be initiated by the canal managers who are mostly engineers. Their incentives include convenience and amenity for good living, career prospects, job status, respectable income in proportion to their calibre, the avoidance of stress due to 'farmers and politicians' complaints and pressures, and professional satisfaction. The motivation of the canal managers is adversely affected by the 'transfer trade' in which operation and maintenance as well as some other postings are sold by politicians (1). The price of such postings may be several times the annual salary of a manager. The manager then raises money from maintenance works and from farmers who seek firm assurance for supply of water. Such corruption on canal irrigation systems has five adverse effects: (i) costs to farmers especially the poorer and weaker, (ii) bad physical work in maintenance, (iii) bad canal management, (iv) indiscipline of field staff, and (v) managers being demoralized and distracted from their proper work (1). Effective vigilance, political reforms and discipline can possibly improve the situation. Besides, separating the operations and maintenance cadres, inculcating awareness among farmers about their rights, and introducing accountability and incentives for managers may effectively improve the conditions to a great extent.

OPERATION AND MAINTENANCE OF CANAL IRRIGATION SYSTEMS

Operation and maintenance of an irrigation and drainage (I and D) system implies management of the system. The plan of operation and management (POM) or the management plan includes a set of documents and instructions, organisation charts, work procedures and rules (including coordination with other disciplines), programmes and schedules which aim at achieving efficient and optimal functioning of the irrigation system. POM is not a rule book but a set of guidelines. POM is not static but dynamic and may require updating during project implementation, post- implementation period of the project (in the light of advances in science and technology, management techniques and the experience gained over the years) in the operation and maintenance of the system.

The operation and maintenance (O and M) unit (or division or designated agency for the formulation of POM) must commence very early in the project planning phase (for aspects such as selection of the conveyance system or determination of the agricultural activities in relation to their costs or training of personnel and farmers) and should continue through the planning, design (for aspects such as detailing the scheme of operation whether regulated/ unregulated supply remote/on-site control, storage requirement etc. ; overall conveyance/delivery system ; the control, monitoring and communication system, specific O and M offices, inspection houses, shops, yards and related features), construction (for installing the O and M organisation in the field, commissioning of project facilities and transferring responsibility from construction to O and M), commissioning, and operation phases.

Performance analysis of major and medium irrigation projects completed in the post-independence period with huge public money has revealed that the contemplated objectives and benefits have not been achieved in several cases. A proper Plan for Operation and Maintenance (29) is, therefore, necessary to achieve stipulated levels of project services including maintenance at minimum achievable cost achieve optimum use of canal water provide detailed O and M guidelines during various anticipated scenarios of water availability, including equitable water distribution upto the tail-end of the system, and

effect efficient coordination of staff, equipment, physical and financial resources and related disciplines, active involvement of farmers etc.

Operation Policy

Operation policy for any irrigation project should be such that the system is operated in the basic interest of beneficiaries i.e., farmers, and policy conforms to all the laws, ordinances of the state.

The project management may frame appropriate rules, proceed with activities for regulation and conservation of water and for the protection of quality of water. Operation within the project will be guided by such rules and operational policies adopted by the project management.

GENERAL

Canal structures include the following types of structures:

Communication structures, such as roads and railways which have to be constructed across the channels. Such structures are in the form of bridges and are not included in this book.

Regulation structures which are meant for controlling discharges, velocity, and water levels in the channels. Canal falls, distributaries head regulators, escapes, etc., are examples of regulation structures.

Cross-drainage structures which are required to pass natural drainage across channels. Aqueducts, siphon aqueducts, siphons, and super passages are examples of cross- drainage structures.

Canal structures listed at (ii) and (iii) above may fail on account of effects of either surface flow or subsurface flow. Water flowing over the structure causes hydrostatic forces, formation of hydraulic jumps, and scour upstream and downstream of the structure. Considerations of subsurface flow are important on all hydraulic structures which have foundations other than that of solid impervious rock. Subsurface flow endangers the stability of hydraulic structures in two ways—by piping and uplift pressure. Piping failure occurs when the seepage water is left with sufficient force to lift up soil particles at the downstream end of a hydraulic structure where it emerges. Uplift pressure is the pressure exerted by the seeping water on a hydraulic structure. If this pressure is not counterbalanced by the weight of concrete or masonry floor, the structure may fail because of rupture of the floor.

HYDRAULIC JUMP

Hydraulic jump occurs when, in the same reach of a channel, the upstream control causes supercritical flow while the downstream control dictates subcritical flow. Hydraulic jump is always accompanied by considerable turbulence and energy dissipation. The following are the useful applications of hydraulic jump:

The dissipation of energy of flow downstream of hydraulic structures such as dams, spillways, weirs, etc.

The reduction of net uplift pressures under hydraulic structures by raising the wa- ter depth on the apron of the structure.

The maintenance of high water levels in channels for water distribution purposes.

The mixing of chemicals for water purification or other purposes (in chemical indus- tries).

On applying Newton's second law of motion to the control volume, shown in Fig. 9.1, one obtains,

$$P_1 - P_2 + W \sin \theta - F_f = \rho Q(\beta_2 u_2 - \beta_1 u_1) \quad (9.1)$$

where P_1 ($= \rho g \bar{z}_1 A_1$) and P_2 ($= \rho g \bar{z}_2 A_2$) are the pressure forces at sections 1 and 2, W the weight of liquid between sections 1 and 2, F_f the component of unknown forces (along the direction of flow) acting between sections 1 and 2, θ the longitudinal slope of the channel, β_1 and β_2 the momentum correction coefficients at sections 1 and 2, u_1 and u_2 are the average velocities at sections 1 and 2 and $\bar{z}_1$ and $\bar{z}_2$ are the distances to centroids of respective flow areas A_1 and A_2 from the free surface.

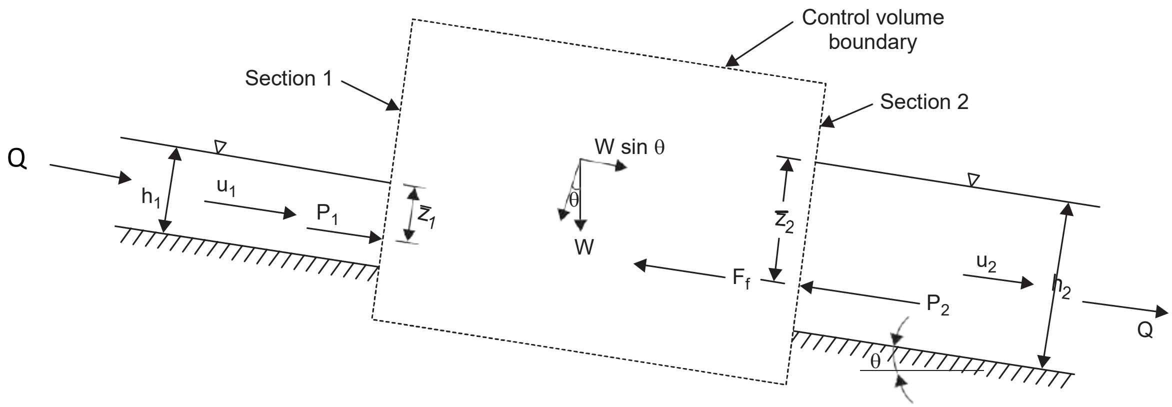


Fig. 9.1. Control volume for hydraulic jump

With the assumptions that θ is small (i.e., $\sin\theta \approx 0$), and $\beta_1 = \beta_2 = 1$, Eq. (9.1) becomes

$$\rho g \bar{z}_1 A_1 - \rho g \bar{z}_2 A_2 - F_f = \rho Q (u_2 - u_1) \qquad (9.2)$$

Equation (9.2) can be rewritten as

$$\frac{F_f}{\rho g} = \left(\frac{Q^2}{gA_1} + A_1 \bar{z}_1\right) - \left(\frac{Q^2}{gA_2} + A_2 \bar{z}_2\right) \qquad (9.3)$$

or

$$\frac{F_f}{\rho g} = M_1 - M_2 \qquad (9.4)$$

where,

$$M = \frac{Q^2}{gA} + A \bar{z} \qquad (9.5)$$

and M is termed the specific momentum or force function or specific force.

If the jump occurs in a horizontal channel and is not assisted by any other means, such as baffle blocks, then $F_f \approx 0$, and Eq. (9.4) yields

$$M_1 = M_2 \qquad (9.6)$$

or

$$\frac{Q^2}{gA_1} + A_1 \bar{z}_1 = \frac{Q^2}{gA_2} + A_2 \bar{z}_2 \qquad (9.7)$$

Hydraulic Jump in Rectangular Channels

A hydraulic jump formed in a smooth, wide, and horizontal rectangular channel is termed classical hydraulic jump. For rectangular channel of width B,

$$Q = u_1 A_1 = u_2 A_2$$

$$A_1 = B_1 h_1 \quad \text{and} \quad A_2 = B_2 h_2$$

$$\bar{z}_1 = h_1/2 \quad \text{and} \quad \bar{z}_2 = h_2/2$$

Substituting these values into Eq. (9.7) one can, after simplification, obtain

$$\frac{1}{2} h_1 h_2 (h_1 + h_2) = \frac{q^2}{g} \qquad (9.8)$$

where, $q = Q/B$. Equation (9.8) has the following solutions :

$$\frac{h_2}{h_1} = \frac{1}{2}\left(\sqrt{1 + 8F_1^2} - 1\right) \qquad (9.9)$$

and

$$\frac{h_1}{h_2} = \frac{1}{2}\left(\sqrt{1 + 8F_2^2} - 1\right) \qquad (9.10)$$

in which, F_1 and F_2 are the Froude numbers at sections 1 and 2, respectively. Froude number F equals $u/\sqrt{gD}$ in which, D is the hydraulic depth and equals A/T where, T is the top width of flow. Equations (9.9) and (9.10) are the well-known Belanger's momentum equations.

Energy Loss in Hydraulic Jump in a Rectangular Channel

In a horizontal rectangular channel with the channel bed chosen as the datum, the total energies at sections 1 and 2, (Fig. 9.1 with $\theta = 0$) are equal to the specific energies E_1 and E_2 at sections 1 and 2, respectively, i.e.,

$$E_1 = h_1 + \frac{q^2}{2gh_1^2} \qquad (9.11)$$

$$E_2 = h_2 + \frac{q^2}{2gh_2^2} \qquad (9.12)$$

so that energy loss, $\Delta E = E_1 - E_2 \qquad (9.13)$

$$= h_1 - h_2 + \frac{q^2}{2g}\left[\frac{1}{h_1^2} - \frac{1}{h_2^2}\right]$$

$$= (h_1 - h_2) + \frac{1}{4} h_1 h_2 (h_1 + h_2)\left(\frac{h_2^2 - h_1^2}{h_1^2 h_2^2}\right)$$

$$= \frac{4h_1^2 h_2 - 4h_1 h_2^2 + h_1 h_2^2 - h_1^3 + h_2^3 - h_1^2 h_2}{4h_1 h_2}$$

$$= \frac{h_2^3 - h_1^3 - 3h_1^2 h_2 + 3h_1 h_2^2}{4h_1 h_2}$$

$$\therefore \quad \Delta E = \frac{(h_2 - h_1)^3}{4h_1 h_2} \qquad (9.14)$$

Also,

$$\frac{\Delta E}{E_1} = \frac{[h_1 + (u_1^2/2g)] - [h_2 + (u_2^2/2g)]}{[h_1 + u_1^2/2g]} \qquad (9.15)$$

$$= \frac{h_1[1 - (h_2/h_1)] + (q^2/2gh_1^2)[1 - (h_1/h_2)^2]}{(h_1/2)[2 + F_1^2]}$$

$$\therefore \quad \frac{\Delta E}{E_1} = \frac{2 - 2(h_2/h_1) + F_1^2[1 - (h_1/h_2)^2]}{2 + F_1^2} \qquad (9.16)$$

Combining Eq. (9.8), (9.11) and (9.14), one can obtain

$$\therefore \quad \frac{\Delta E}{E_1} = \frac{8F_1^4 + 20F_1^2 - (8F_1^2 + 1)^{3/2} - 1}{8F_1^2 (2 + F_1^2)} \qquad (9.17)$$

Hence, for a supercritical Froude number F_1 equal to 20, the energy loss ΔE is equal to 0.86 E_1. This means that 86 per cent of the initial specific energy is dissipated. Because of this energy dissipating capability, hydraulic jump is widely used as an energy dissipator for spillways and other hydraulic structures. In a hydraulic jump, the mean kinetic energy is first converted into turbulence and then dissipated through the action of viscosity. Equation (9.17) can be rewritten as

$$\therefore \quad \frac{E_2}{E_1} = \frac{(8F_1^2 + 1)^{3/2} - 4F_1^2 + 1}{8F_1^2 (2 + F_1^2)} \qquad ...(9.18)$$

The term E_2/E_1 is called the efficiency of the jump.

Length of Hydraulic Jump

The length of a hydraulic jump is defined as the distance from the front of the jump to a section of the flow immediately downstream of the roller associated with the jump. Although a very important design parameter, the length of a hydraulic jump, L_j cannot be derived from theoretical considerations. The length of a hydraulic jump is approximately equal to five times the height of the jump which itself is $(h_2 - h_1)$. Silvester (1) has shown that for horizontal rectangular channels, the ratio L_j/h_1 is a function of the upstream supercritical Froude number.

He obtained,

$$\frac{L_j}{h_1} = 9.75\,(F$$

$- 1)^{1.01}$ (9.19)

h_1 1

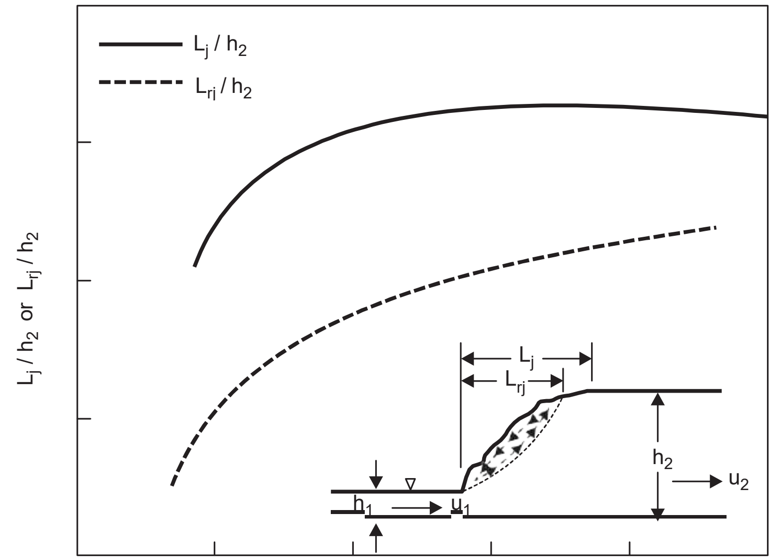
The length of a classical hydraulic jump L_j can also be estimated from Fig. 9.2 which shows the variation of L_j/h_2 with F_1.

8

6

4

2



0 F_1 7 9 11

1 3 5

Fig. 9.2 Length characteristics of jump (2)

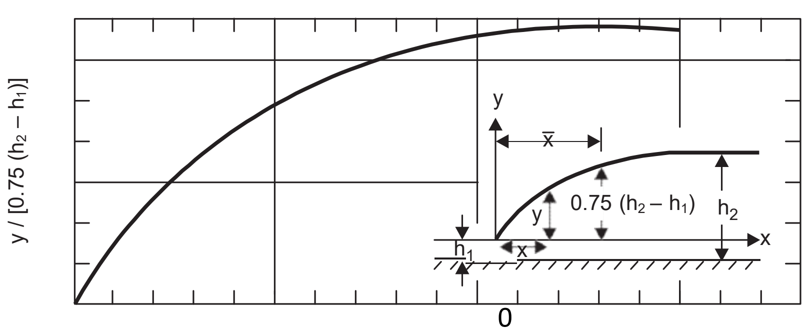
Profile of Hydraulic Jump

In case of overflow structures located on permeable foundations, the concrete aprons of the stilling basins are subjected to uplift pressures which are partly counterbalanced by the weight of water flowing on the apron. Therefore, in the hydraulic jump type stilling basins, determination of profile of the jump becomes necessary. Rajaratnam and Subramanya (3) have obtained an empirical relation, (Fig. 9.3), between $y/[0.75(h_2 - h_1)]$ and $x/\bar{X}$. Here, $\bar{X}$ is the distance from the beginning of the jump to the section where the depth measured above the x-axis is $0.75(h_2 - h_1)$. The length $\bar{X}$ is empirically related to h_1 and F_1 as

$$\frac{\bar{X}}{h_1} = 5.08 F_1 - 7.82 \quad (9.20)$$

1.2

0.6



0 1.0 2.0 –

x / X

3.0

Fig. 9.3 Profile of the hydraulic jump in a rectangular channel (3)

Calculations for Hydraulic Jump in Horizontal Rectangular Channels

Equations (9.8), (9.11), (9.12), and (9.14) can be used to obtain direct solution of the jump parameters (i.e., E_2, h_2, $\square E$, and E_1) for horizontal rectangular channels, if h_1 and q are known. These equations would also yield direct solution for E_1, h_1, $\square E$, and E_2, if h_2 and q are known.

One can also use Eq. (9.9) or Eq. (9.10) instead of Eq. (9.8) depending upon whether pre-jump or post-jump conditions are known.

However, in an actual design problem, generally the discharge q and the levels of the upstream and downstream total energy lines are known. Thus, q and $\square E$ are known. Determination of the remaining four parameters of the jump from Eqs. (9.8), (9.11), (9.12), and (9.14) is rather difficult. This difficulty can be overcome by the use of critical depth h_c (= $(q^2/g)^{1/3}$) and defining

$X = (h_1/h_c)$; $Y = (h_2/h_c)$; $Z = (\square E/h_c)$;

$\square = E_1/h_c$, and $\square = E_2/h_c$

so that Eqs. (9.8), (9.11), (9.12), and (9.14) reduce to the following forms respectively :

$$XY(X + Y) = 2 \qquad (9.21)$$

$$\square = X + \frac{1}{2X^2} \qquad (9.22)$$

$$\square = Y + \frac{1}{2Y^2} \qquad (9.23)$$

and $$Z = \frac{(Y \square X)^3}{4XY} \qquad (9.24)$$

Here, X can vary from 0 to 1 only. Using Eqs. (9.21) to (9.24), one can obtain the sets of values of Y, $\square$, $\square$ and Z for different values of X and, thus, the curves shown in Fig. 9.4. These curves are known as Crump's curves (4). The method to use these curves is as follows:

Calculate h_c from $h_c = (q^2/g)^{1/3}$

Compute Z, i.e., $\square E/h_c$.

Read h_2/h_c from the curve $\square E/h_c$ versus h_2/h_c.

Read E_2/h_c from the curve E_2/h_c versus h_2/h_c.

Thus, $E_1 = \square E + E_2$.

For known E_1/h_c, obtain h_1/h_c from the curve E_1/h_c versus h_1/h_c.

Thus, for given q and $\square E$, one can determine h_2, E_2, E_1, and h_1 from the relationships for Crump's coefficients.

Combining Eqs. (9.21) and (9.24), one can obtain (5),

$$Z = \frac{\square X^6 \square 20X^3 \square 8 \square (X^4 \square 8X)^{3/2}}{16X^2} \qquad (9.25)$$

and also $$Z = \frac{\square Y^6 \square 20Y^3 \square 8 \square (Y^4 \square 8Y)^{3/2}}{16Y^2} \qquad (9.26)$$

For different values of X and Y, the values of Z can be obtained and the curves Y-Z prepared. Approximate equations for these curves were obtained as (5)

$$Y = 1 + 0.93556\, Z^{0.368} \text{ for } Z < 1 \qquad (9.27)$$

and $$Y = 1 + 0.93556\, Z^{0.240} \text{ for } Z > 1 \qquad (9.28)$$

1.4

1.2

1.0

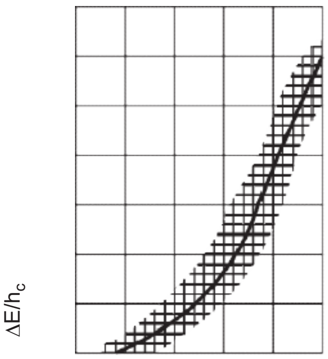
0.8

0.6

0.4

0.2

0



1.0 1.2 1.4 1.6 1.8 2.0

h_2/h_c (Subcritical, enlarged scale)

7.0

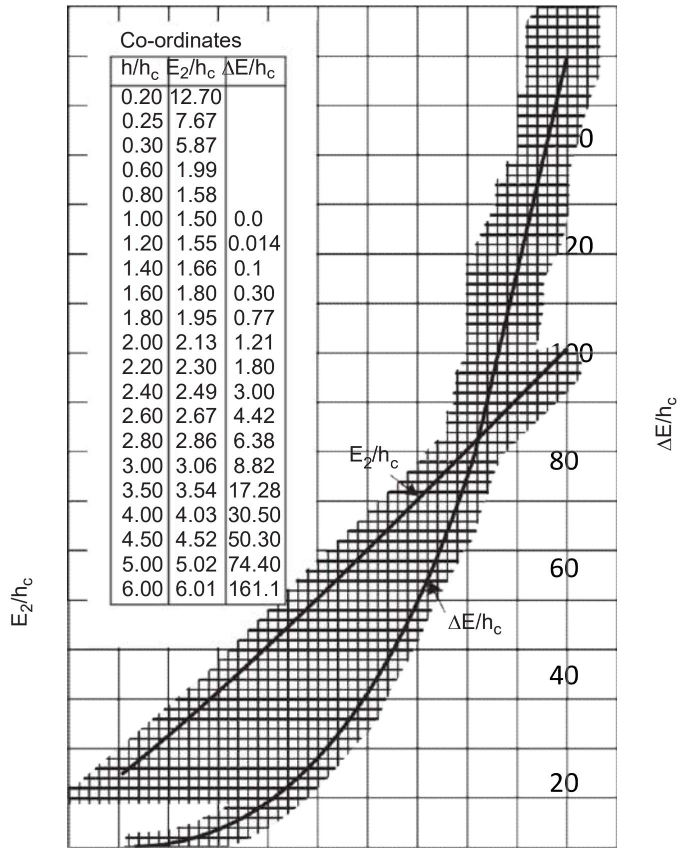
6.0

5.0

4.0

3.0

2.0



0

20

00

80

60

40

20

14.0

12.0

10.0

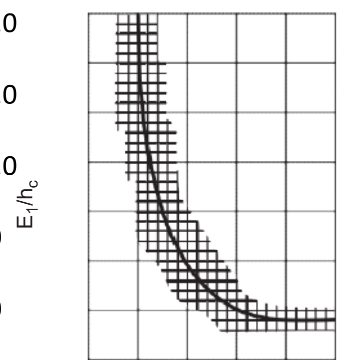
8.0

6.0

4.0

2.0

0.0



0.0 0.2 0.4 0.6 0.8 1.0 1.0

1.0 2.0 3.0 4.0 5.0 6.0

0

h_1/h_c (Supercritical) h_2/h_c (Subcritical)

Fig. 9.4 Relationship for Crump's coefficients (4)

One of the two equations (Eqs. (9.27) and (9.28)) can be used for obtaining the value of Y for a specified value of Z. Equations (9.21), (9.22), and (9.23) can then be used for the determination of X, ?, and ? respectively.

Equations (9.25) and (9.26), can be solved to prepare a table for computations of hydraulicjump elements in rectangular channels (Table 9.1).

Table 9.1 Hydraulic jump elements in rectangular channels (5)

Z	X	Y	$\frac{E_1}{h_c}$	$\frac{E_2}{h_c}$	Y [illegible] $\frac{h_2}{h_1}$	X[illegible]$E E_1$
0.01	0.839	1.180	1.549	1.539	1.406	0.006
0.10	0.681	1.407	1.760	1.660	2.067	0.057
0.50	0.516	1.728	2.396	1.896	3.351	0.209
1.00	0.436	1.936	3.069	2.069	2.442	0.326
1.50	0.389	2.082	3.700	2.200	4.356	0.406

(Contd.)...

2.00	0.356	2.199	4.303	2.303	6.179	0.465
2.50	0.331	2.298	4.893	2.393	6.940	0.511
3.00	0.311	2.384	5.472	2.472	7.659	0.548
4.00	0.281	2.531	6.609	2.609	9.002	0.605
4.50	0.269	2.594	7.169	2.669	9.638	0.628
5.00	0.259	2.654	6.725	2.725	10.254	0.647
6.00	0.241	2.761	8.826	2.826	11.439	0.680
7.00	0.227	2.856	9.917	2.917	12.572	0.706
8.00	0.215	2.942	11.000	3.000	13.663	0.727
10.00	0.197	3.094	13.146	3.146	15.744	0.761
15.00	0.165	3.396	18.439	3.439	20.527	0.814
20.00	0.145	3.640	23.878	3.878	25.079	0.846

Hydraulic Jump on Sloping Channels

On a horizontal floor with little friction, location of hydraulic jump varies considerably with a slight change in the depth or velocity of flow. But, on a sloping floor, location of hydraulic jump is relatively stable and can be closely predicted. However, energy dissipation in the case of jumps on a sloping floor is less owing to the vertical component of velocity remaining intact.

Equation (9.1) is theoretically applicable to hydraulic jumps forming on sloping channels.

But the solution of the problem is difficult due to the following reasons :

The length and shapes of the hydraulic jump are not well-defined and, hence, the term W sin θ is poorly computed.

The specific weight of the liquid in the control volume can change considerably due to air entrainment.

The pressure terms cannot be determined accurately.

Figure 9.5 shows several cases of hydraulic jumps on sloping channels. In the studies of hydraulic jump on sloping channels, the end of the surface roller is taken as the end of the jump. This means that the length of roller (measured horizontally) is the length of the jump.

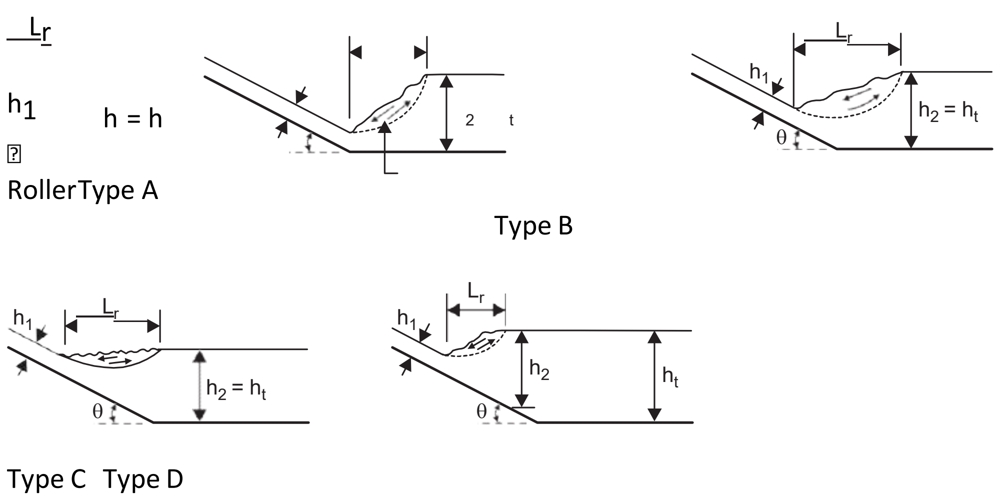


Fig. 9.5 Different types of hydraulic jump which form in sloping channels

When the jump begins at the end of the sloping apron, type A jump occurs and $h_2 = h_2^* = h_t$. Here, h_2 is the subcritical sequent depth corresponding to h_1, h_t the tail-water depth, and h_2^* is the subcritical sequent depth h_2 given by Eq. (9.9). Type A jump is, obviously, governed by Eqs. (9.9) and (9.10).

When the end of the jump coincides with that of the sloping bed, type C jump occurs. For this case, Kindsvater (6) developed the following equation for the sequent depth h_2 :

$$\frac{h_2}{h_1} = \frac{1}{2\cos\theta}\left[\sqrt{1 + 8F_1^2\,\frac{\cos^3\theta}{1 - 2N\tan\theta}} - 1\right] \quad (9.29)$$

in which, θ is the longitudinal slope angle of the channel, and N is an empirical coefficient dependent on the length of the jump. Equation (9.29) can be rewritten as

$$\frac{h_2}{h'_1} = \frac{1}{2}\left[\sqrt{1 + 8G_1^2} - 1\right] \quad (9.30)$$

where, $h'_1 = h_1/\cos\theta$ (9.31)

$$\text{and} \quad G_1^2 = \frac{\cos^3\theta}{1 - 2N\tan\theta} \times F_1^2 \quad (9.32)$$

Rajaratnam (6) gave the following simple expression :

$$\frac{\cos^3\theta}{1 - 2N\tan\theta} = 10^{0.054\,(\theta)} \quad (9.33)$$

where, θ is in degrees.

When h_t is greater than the sequent depth h_2 required for type C jump, then type D jump occurs completely on the sloping apron. Bradley and Peterka (7) found that Eqs. (9.30) to (9.33) valid for type C jump can be used for the type D jump also.

If h_t is less than that required for type C jump but greater than h_2^*, the toe of the jump is on the sloping bed, and the end of the jump on the horizontal bed. This jump is classed as type B jump. A graphical solution (Fig. 9.6) has been developed for this type of jump (7).

Bradley and Peterka (7) have developed plots for the estimation of the length of the type D jump (Fig. 9.7). These plots can also be used to determine the lengths of the types B and C jumps.

The energy loss for the type A jump can be estimated from Eq. (9.14). For C and D jumps, one can write

$$E_1 = L_j\tan\theta + \frac{h_1}{\cos\theta} + \frac{u_1^2}{2g} \quad (9.34)$$

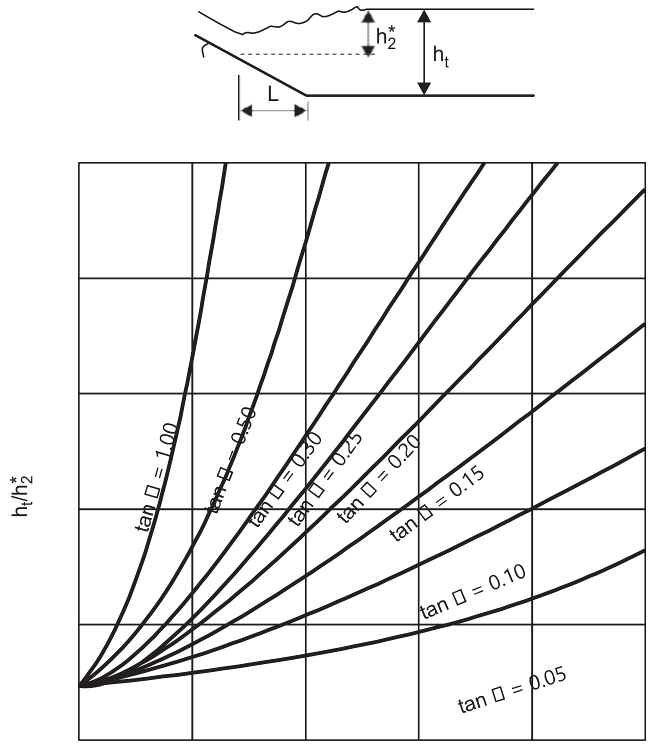
$$\text{and} \quad E_2 = h_2 + \frac{u_2^2}{2g}$$

(9.35)

Thus,

$$\frac{\Delta E}{E_1} = \frac{(1 - h_2/h_1) + (F_1^2/2)\,[1 - \{1/(h_2/h_1)\}^2] + [(L_j/h_2)\,(h_2/h_1)]\tan\theta}{1 + (F_1^2/2) + (L_j/h_2)\,(h_2/h_1)\tan\theta} \qquad (9.36)$$

□



2.8

2.4

2.0

1.6

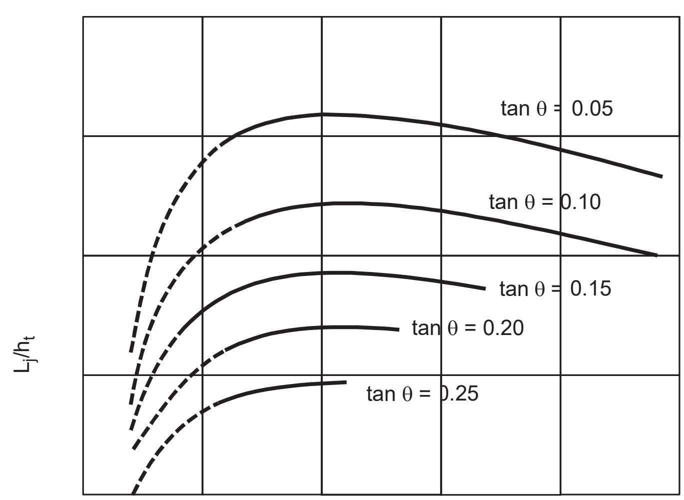
1.2

0.8

0 2 4 6 8 10

L/h_2^*

Fig. 9.6 Solution for B jump (2)



6

5

4

3

2

0 4 8 12 16 20

u_1

$F_1 = \frac{u_1}{\sqrt{gh_1}}$

Fig. 9.7 Hydraulic jump length for jump types B, C and D (7)

Here, the bed level at the end of the jump has been chosen as the datum and the potential energy term $h_1/\cos\theta$ has been approximated as h_1. Equation (9.36) should not be used when F_1 is less than 4 as in this range very little is known about L_j/h_2 which affects $\Delta E/E_1$. In order to solve a problem of hydraulic jump on a sloping channel, the first step is to determine the type of jump for given slope, the pre-jump supercritical depth, and the tail-water condition, Figure 9.8 illustrates the procedure for the determination of the type of hydraulic jump.

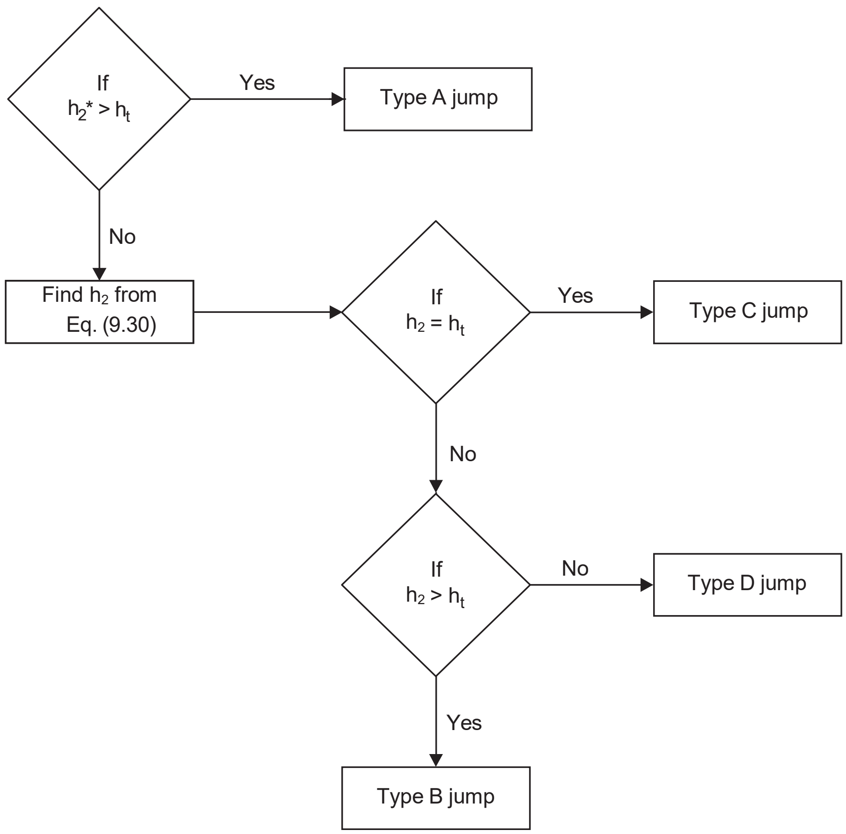


Fig. 9.8 Determination of type of hydraulic jump on sloping channels

Example 9.1 A discharge of 9.0 m^3/s flows in a 6.0 m wide rectangular channel which is inclined at an angle of 3° with the horizontal. Determine the type of jump if h_1 = 0.10 m and h_t = 2.6 m.

Solution:

$$F_1 = \frac{9.0/(6.0 \times 0.1)}{\sqrt{9.81 \times 0.10}} = 15.15$$

h_2* (i.e., the sequent depth in a horizontal channel) can be calculated from

$$h_2^* = \frac{h_1}{2}\left[\sqrt{1+8F_1^2} - 1\right] = \frac{0.1}{2}\left[\sqrt{1+8(15.15)^2} - 1\right]$$

= 2.09 m

Since $h_t > h_2^*$, the depth h_2 should be calculated from Eqs. (9.30)-(9.33).

$$G_1^2 = 10^{0.054(\theta)} F_1^2$$

$$= 10^{0.054(3)} \times (15.15)^2$$

$$= 333.29$$

$$\therefore \quad \frac{h_2}{(0.10/\cos 3°)} = \frac{1}{2}\left[\sqrt{1 + 8\,G_1^2} - 1\right]$$

or $$h_2 = \frac{0.10}{2\cos(3°)}\left[\sqrt{1 + 8(333.29)} - 1\right] = 2.54 \text{ m}$$

Since $h_2 < h_t$, the jump is classified as the type D jump. Using Fig. 9.7, the length of the jump, L_j, can be determined by obtaining L_j / h_t for tan 3° = 0.05 and F_1 = 15.15.

$$\frac{L_j}{h_t} = 4.9$$

$$\therefore \quad L_j = 4.9 \times 2.6 = 12.74 \text{ m}$$

Now $$\frac{h_t}{h_1} = \frac{2.6}{0.1} = 26$$

and $$\frac{h_2}{h_1} = \frac{2.31}{0.1} = 23.1$$

Therefore, relative energy loss can be determined from Eq. (9.36)

$$\frac{\Delta E}{E_1} = \frac{(1 - 23.1) + [(15.15)^2/2]\,[1 - (1/23.1)^2] + \tan(3°)\,[4.9(26)]}{1 + [(15.15)^2/2] + (4.9)(26)\tan(3°)} = 0.82$$

Forced Hydraulic Jump

When the tail-water depth h_t is less than the required sequent depth h_2 corresponding to the pre-jump depth h_1, the jump is repelled downstream. However, by introducing devices such as baffle walls or baffle blocks and, thus, increasing the surface friction, the jump can be made to move upstream and form forcibly at the section it would have formed if sufficient tail-water depth (= h_2) were available. Such a jump is called the forced hydraulic jump (Fig. 9.9).

Figure 9.10 shows different types of forced hydraulic jump (8). For small Z and large x_o, type I jump is formed. This is similar to a free jump. When Z is increased and x_o is decreased, the baffle acts like an obstruction placed across the channel having free flow conditions and the jump is of type II*. With increase in tail-water depth, the obstruction is submerged and type II jump forms. On increasing Z and decreasing x_o further, the jump becomes more violent and is of type III. The effect of further increase in Z and decrease in x_o results in jumps of type IV, VI and VI*. A type VI* jump is a type VI jump with low tail-water depth. Between types IV and VI, there occurs an unstable transition phenomenon called type V (not shown in Fig. 9.10).

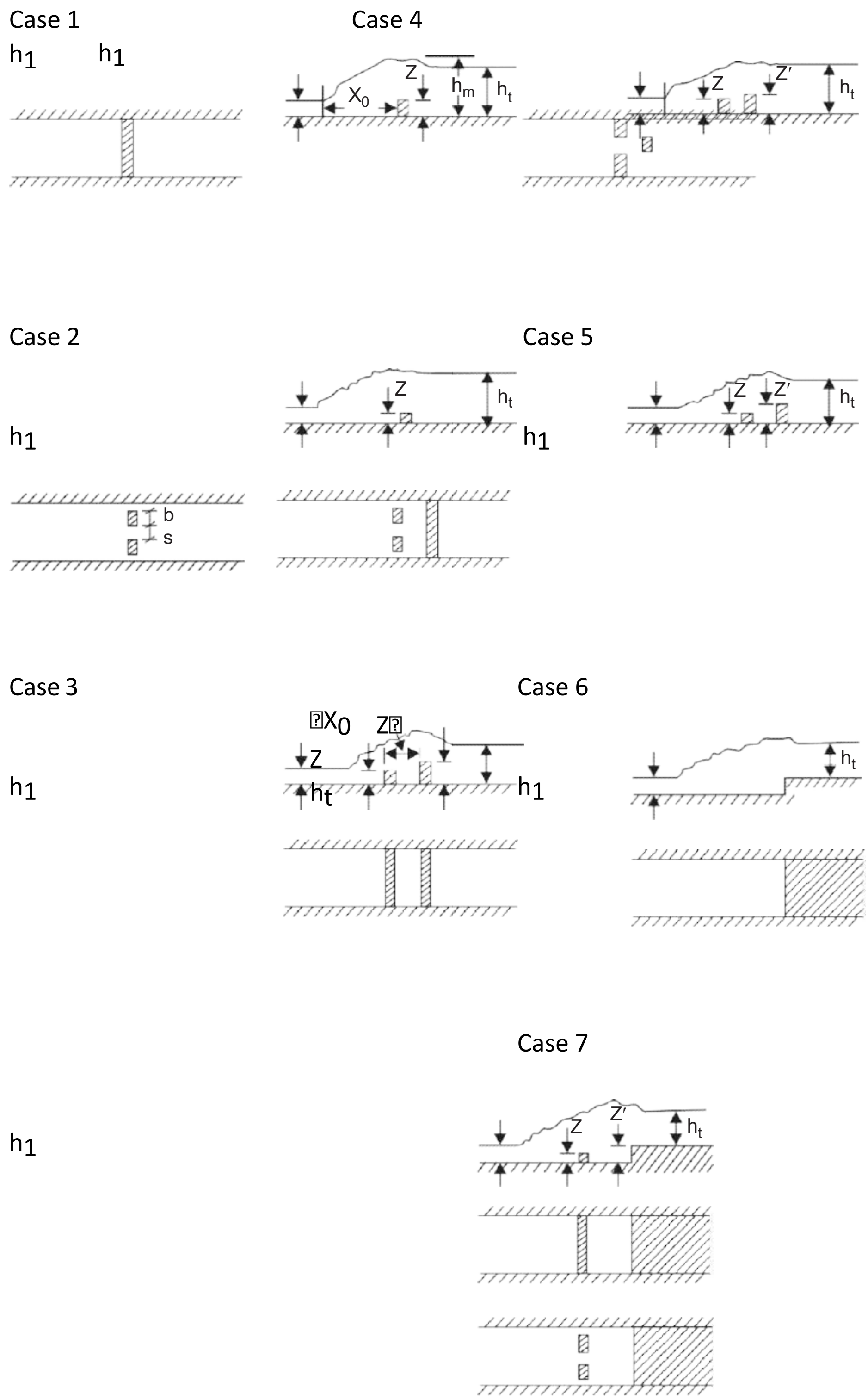


Fig. 9.9 Devices for producing forced hydraulic jump (8)

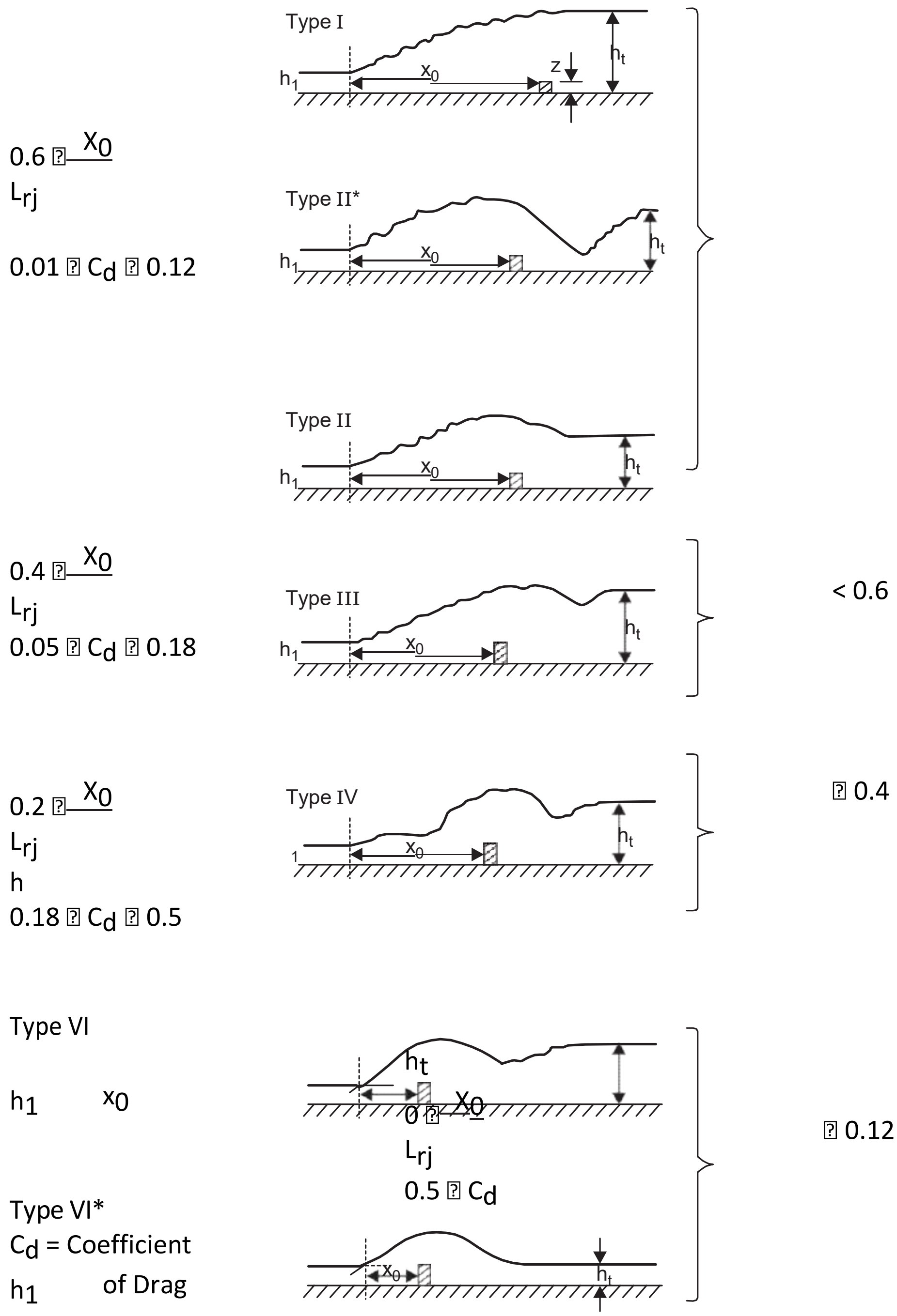


Fig. 9.10 Types of forced hydraulic jump (8)

A large number of experimental studies have been carried out on the forced hydraulic jump as it forms the basic design element of the well-known hydraulic jump type stilling basins. The simplest case of a forced hydraulic jump is the jump forced by a two-dimensional baffle, known as a baffle wall, of height Z kept at a distance x_o from the toe (i.e., the beginning) of the jump (Fig. 9.9, case 1).

Considering unit width of the channel, the momentum equation can be written as

$$\frac{1}{2}\rho g h_1^2 - \frac{1}{2}\rho g h_t^2 - F_b = \rho q\left(\frac{q}{h_t} - \frac{q}{h_1}\right) \qquad (9.37)$$

where, $F_b = C_d(\rho u_1^2/2)\, Z$.

Equation (9.37), on simplifying, gives

$$C_{d1} = \frac{(\alpha - 1)\,[2F_1^2 - \alpha(1+\alpha)]}{F_1^2\,\alpha\,\beta} \qquad (9.38)$$

where, $\alpha = h_t/h_1$ and $\beta = Z/h_1$. Rajaratnam (8) found experimentally that C_d obtained from Eq. (9.38) is a function of only x_o which is made dimensionless by the length of roller of the classical jump, L_{rj} (Figs. 9.2 and 9.11). Figure 9.11 is based on data from only one source and, therefore, needs further verification. A design chart (Fig. 9.12) has been developed (8), using Eq. (9.38), with γ (= h_t/h_2 = $(h_t/h_1)(h_1/h_2)$ = α/η where, $\eta = h_2/h_1$) versus F_1 for various values of βC_d. Choosing a C_d value of 0.4 (a very competitive design) or smaller (for a conservative design), obtain the value of $C_d \beta$ (and, hence, β) from Fig. 9.12, for known γ and F_1. Now obtain x_o/L_{rj} from Fig. 9.11. Using Fig. 9.2, L_{rj} (and, hence, x_o) can now be determined. From known β and h_1, the depth of baffle wall Z can also be determined.

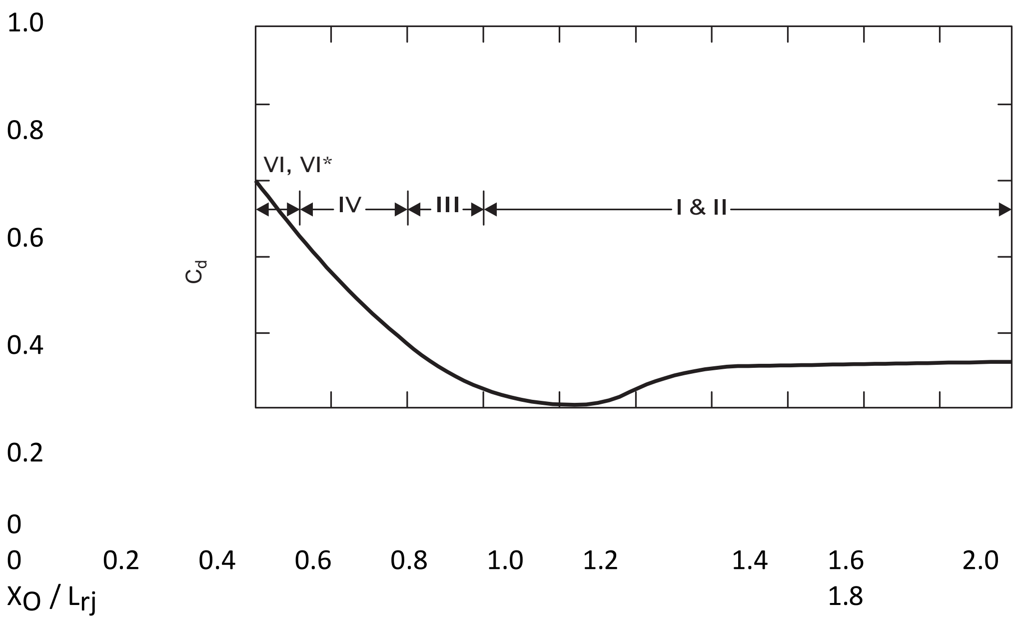


Fig. 9.11 Variation of the drag coefficient for forced hydraulic jump (8)

Baffle block (also known as baffle pier or friction block) is the case of a three-dimensional baffle wall. Baffle blocks are generally trapezoidal in shape and are placed in a single row or in two rows with staggered pattern (Fig. 9.13). The momentum equation [Eq. (9.37)] is applicable for this case too but with a different expression for F_b (the force exerted by the baffle blocks per unit width of the channel) which can be written as

$$\frac{F_b}{F_2} = f\left(\frac{x_o}{h_1}, \alpha, F_1, \beta\right) \qquad (9.39)$$

Here, $F_2 = \frac{1}{2} \, \square \, g h_2^2$ and $\square$ (i.e., the blockage ratio) = W/(W + S) (Fig. 9.13). Based on the analysis of data of Basco and Adams (10), Ranga Raju et al. (9) found that F_1 is unimportant in Eq. (9.39) and ⍰$_1$⍰$_2 F_b/F_2$ is uniquely related to x_o/h_1 as shown in Fig. 9.14. ⍰$_1$ and ⍰$_2$ are empirical correction factors and are functions of Z/h_1 and ⍰, respectively, as shown in Figs. 9.15 and 9.16.

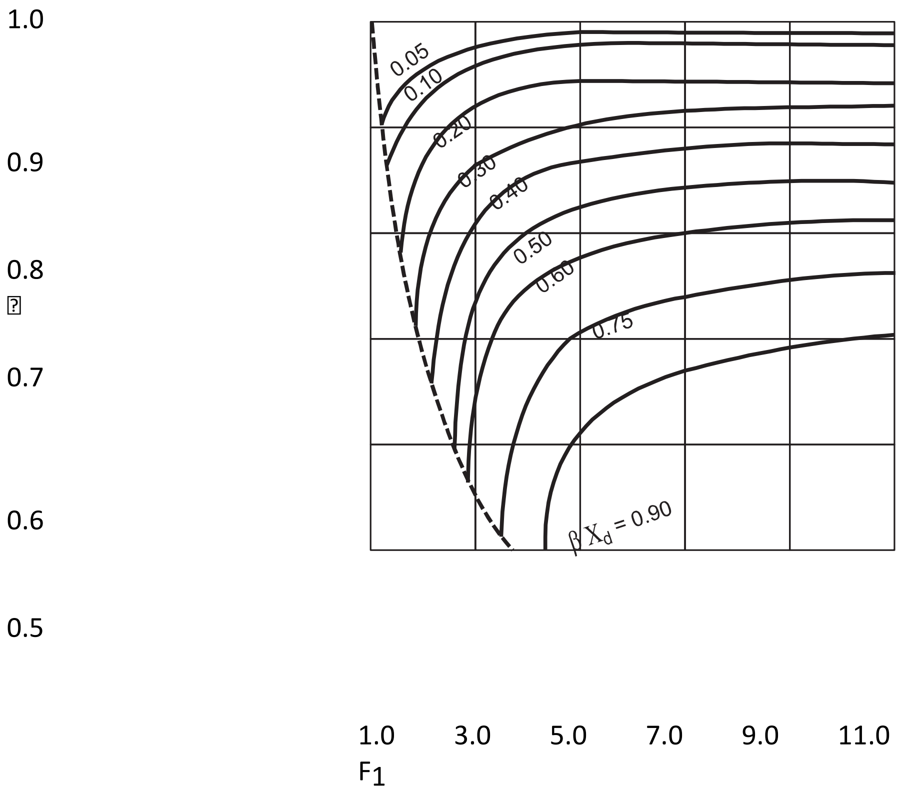
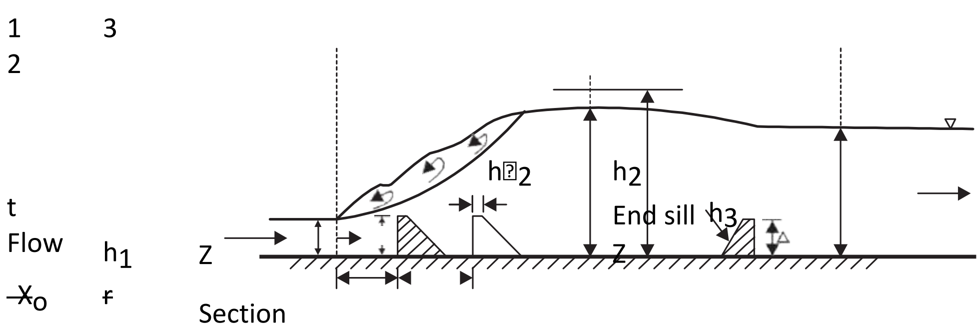
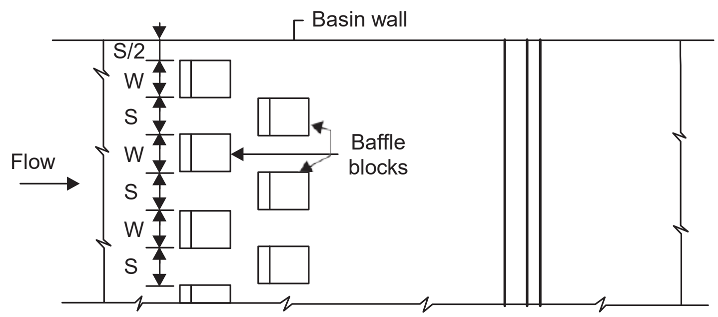


Fig. 9.12 Design chart for baffle wall (9)





Plan

Fig. 9.13 Arrangement of trapezoidal baffle blocks

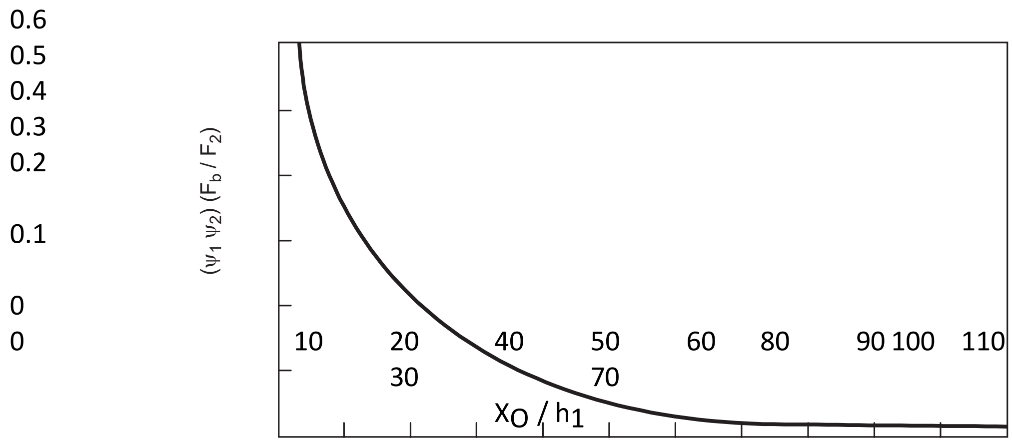


Fig. 9.14 Variation of ($\psi_1\psi_2$ F_b/F_2) with x_o/h_1 for trapezoidal blocks (9)

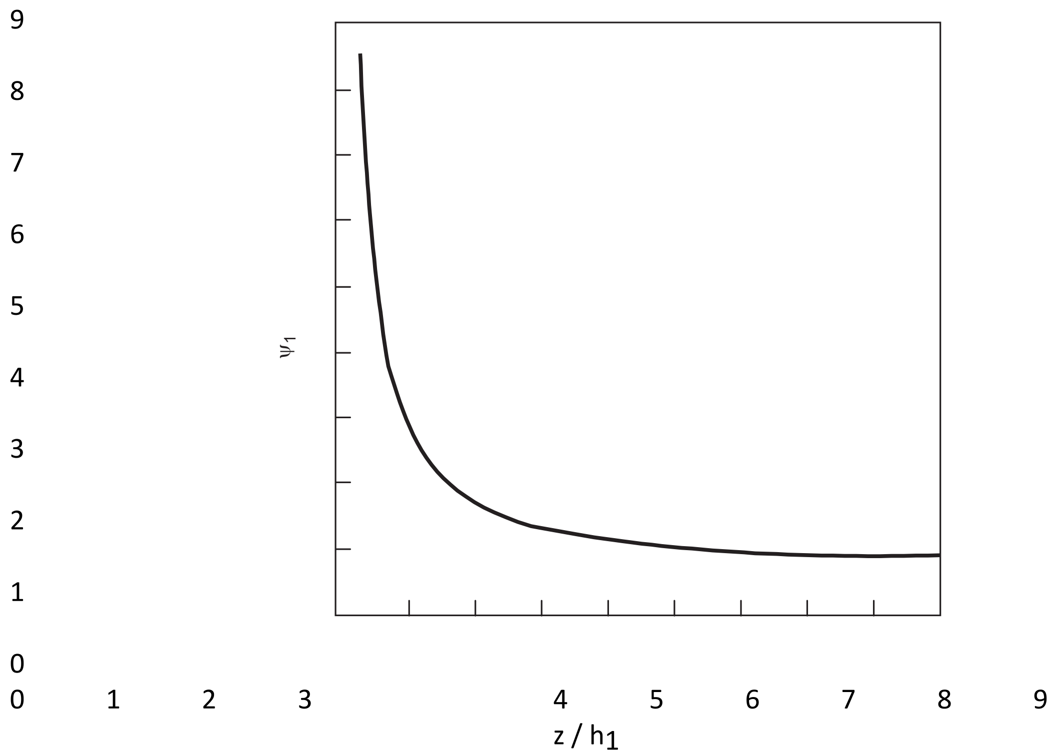


Fig. 9.15 Variation of ψ_1 with Z/h_1 (9)

These data also indicated no change in the value of F_b when baffle blocks were placed in two rows for the range of r/Z from 2.5 to 5.0. Here, r is the spacing between the two rows of blocks.

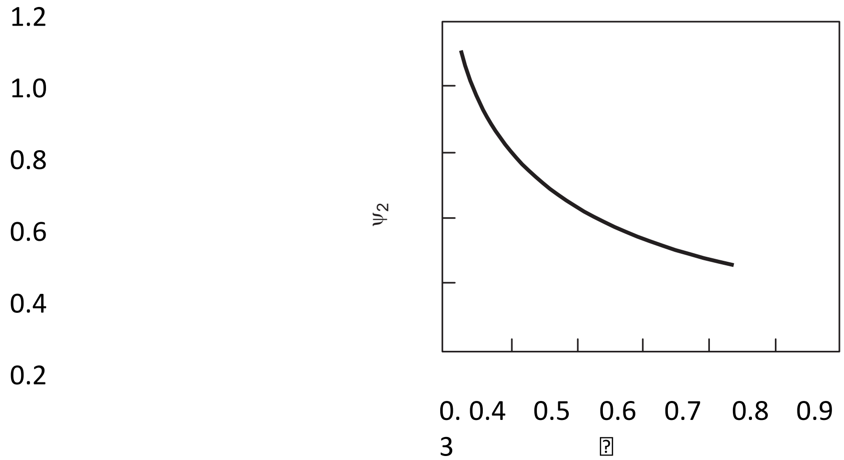


Fig. 9.16 Variation of ψ_2 with □ (9)

Location of Hydraulic Jump on Glacis

It is generally assumed that the jump forms at the junction of the glacis with the horizontal floor and, therefore, the relations for classical hydraulic jump are used for determining the location of the jump on glacis. For known discharge intensity q, and the difference in the levels of the upstream and downstream levels of total energy line, ΔE, one can determine the downstream specific energy, E_2, using a suitable method or Blench's curves (Fig. 9.17). The location of the jump on a glacis can then be obtained by finding the intersection of the glacis with the horizontal plane E_2 below the downstream level of the total energy line.

SEEPAGE FORCE

All the canal structures constructed in India up to the 19th century were mainly designed on the basis of the experience and intuition of the designer. The canal structures, not founded on solid rock, have the problem of seepage. Henry Darcy (11) was the first to propose an experimental relationship [Eq. (4.1)] for flow of water through porous medium. In 1895, following the damage to the Khanki weir on Chenab river a test program was initiated (12). These experiments proved the validity of Darcy's law. The experiments and observations on the Narora weir confirmed that the piping effect and uplift pressure due to seepage endanger the stability of structure founded on a permeable foundation.

Bligh (13) gave a simple method to calculate uplift pressure below a masonry or concrete structure. He stated that the length of the path of flow had the same effectiveness in reducing uplift pressures irrespective of the direction of flow. The length of the flow path along the bottom profile of the structure, i.e., length of the uppermost flow line, was termed creep length. Thus, Bligh made no distinction between horizontal creep and vertical creep. According to Bligh's theory, the creep length (i.e., the length of flow path) L in an idealised weir profile (Fig. 9.18) is equal to $2d_1 + b_1 + 2d_2 + b_2 + 2d_3$. Thus, the loss of head per unit creep length is H/L which is the average hydraulic gradient. Here, H is the seepage head. Bligh termed the average hydraulic gradient as the percolation coefficient C and assigned safe values to it for different types of soil. For coarse-grained soil, the value of C is taken as 1/12 while for sand mixed with boulder and gravel, and for loam soil, the value of C may vary from 1/5 to 1/9. For fine micaceous sand of north Indian rivers, the value of C is 1/15. According to Bligh, if the average hydraulic gradient is less than the assigned safe value of C, there will be no danger of piping.

6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0

0.5
0 0.5 1.0
ΔE (Metres)

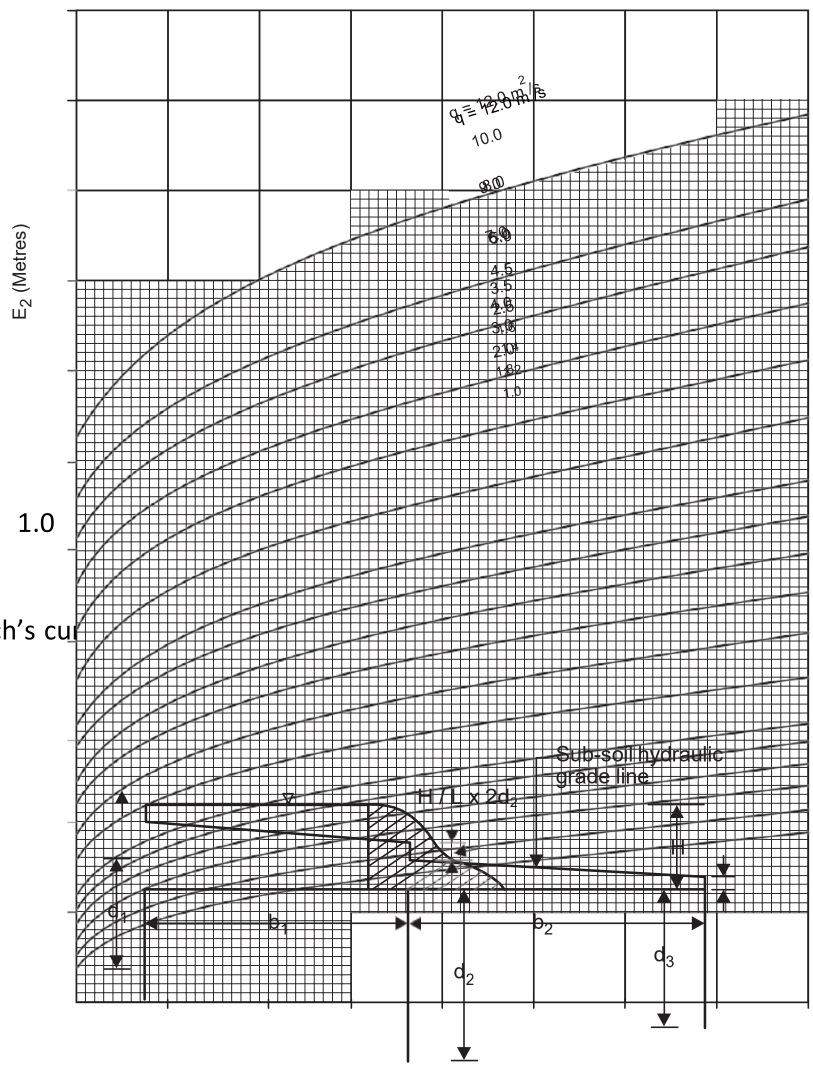


Fig. 9.17 Blench's cu

H/L x $2d_1$

H/L x $2d_3$

Fig. 9.18 Subsoil hydraulic grade line

It should be noted that the seepage head H is the difference between water levels upstream and downstream of the weir. The worst condition will occur when water is held up to the highest possible level on the upstream side with no flow. The downstream level is then taken as the downstream bed level.

The elevation of the subsoil hydraulic gradient line above the bottom of the floor at any point measures the uplift pressure at that point. If h' is the height of the subsoil hydraulic gradient line above the bottom of the floor, the uplift pressure of subsoil water exerted on the floor is $\rho g h'$. Assuming the mass density of the floor material as ρ_s and the floor thickness as t, the downward force per unit area due to the weight of the floor is $t\rho_s g$. For equilibrium, these two should be equal.

$$\therefore \quad \rho g h' = t \rho_s g \quad (9.40)$$

$$\text{or} \quad t = \frac{h'}{(\rho_s/\rho)}$$

The surface profile of the floor is determined from the surface flow considerations and is known. But h', measured from the bottom of the floor, can be known only if the thickness of the floor t is known. Equation (9.40) is, therefore, rewritten as

$$h' = t(\rho_s/\rho)$$

Subtracting t from both sides,

$$h' - t = t(\rho_s/\rho) - t$$

$$= t[(\rho_s/\rho) - 1]$$

$$\therefore \quad t = \frac{h' - t}{(\rho_s/\rho) - 1} \quad (9.41)$$

$(h' - t)$ is the height of subsoil hydraulic gradient line measured above the top surface of the floor and, hence, is known. The thickness of the floor t can, therefore, be directly obtained.

On the upstream side of the barrier, the weight of water causing the seepage is more than sufficient to counterbalance the uplift pressure. In the absence of water upstream, there will be no seepage and, hence, no uplift pressure. Therefore, the upstream floor thickness may be kept equal to minimum practical thickness to resist wear and development of cracks. The floor downstream of the barrier must be in accordance with Eq. (9.41). This also suggests that it would be economical to keep as much floor length upstream as possible. A minimum floor length downstream of the barrier is, however, always required to resist the action of fast-flowing water. Shifting of the floor upstream also reduces the uplift pressure on the floor downstream of the barrier (Fig. 9.19) because a larger portion of the total loss of head has occurred up to the barrier. Further, an upstream cutoff reduces uplift pressure (Fig. 9.20) while a downstream cutoff increases uplift pressure all along the floor.

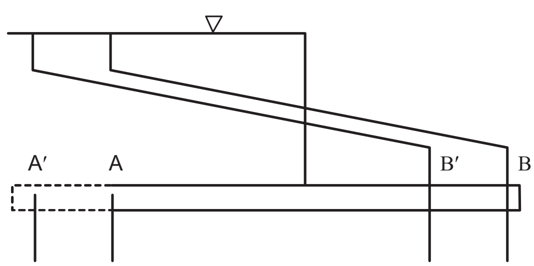


Fig. 9.19 Effect of shifting floor upstream on subsoil H.G.L.

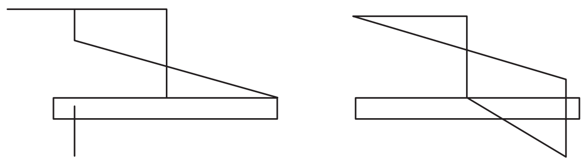


Fig. 9.20 Effect of U/S and D/S cutoffs on subsoil H.G.L

In 1932, Lane (14), after analysing 290 weirs and dams, evolved what is now known as weighted creep theory which, in effect, is Bligh's creep theory corrected for vertical creep. When the coefficient of horizontal permeability is three times the coefficient of vertical permeability, Lane suggested a weight of three for the vertical creep and one for the horizontal creep. Thus, for the case of Fig. 9.18, according to Lane's method, the creep length is $[3(2d_1 + 2d_2 + 2d_3) + (b_1 + b_2)]$. Inclined floors may be treated as vertical if its slope exceeds 45° and horizontal if the slope is less than 45°. Alternatively, for inclined floors, the weight may be taken as equal to $3[1 + (2\theta/90)]$ where θ is the angle (in degrees) of inclination of the floor with the horizontal. Because of their simplicity, Bligh's and Lane's methods are still useful for preliminary dimensioning of the floor. Bligh's method is, obviously, more conservative.

Following the appearance of cracks at the upstream and downstream ends due to the undermining of soil at the upper Chenab canal structures in 1926-27, Khosla, et al. (12) carried out some studies. These studies disclosed that the measured pressures were not equal to those calculated from Bligh's theory. Other notable findings of these studies (12) were as follows :

The outer faces of the end sheet piles were more effective than the inner ones and the horizontal length of floor.

The intermediate piles, if smaller in length than the outer ones, were ineffective except for local redistribution of pressures.

Erosion of foundation soil below the structure started from the tail end. If the hy- draulic gradient at the exit was more than the critical gradient for the soil below the structure, the soil particles would move with the flow of water thus causing progres- sive degradation of the soil and resulting in cavities and ultimate failure.

It was absolutely essential to have a reasonably deep vertical cutoff at the down- stream end to prevent undermining.

In 1929, Terzaghi (15), based on his laboratory studies stated that failure occurred due to undermining if the hydraulic gradient at the exit was more than the floatation gradient. The floatation gradient is similar to the term 'critical gradient' used by Khosla, et al. (12). The floatation gradient implied a state of floatation of the soil at the toe of the work if the exit gradient exceeded the limit of 1 : 1 at which condition the upward force due to the flow of water was almost exactly counterbalanced by the weight of the soil.

The above-mentioned methods of determining seepage effects do not have any theoretical justification. Now that considerable knowledge of the theory of seepage is available, the equation governing the seepage flow has been solved to obtain the uplift pressures and the exit gradient.

Theory of Seepage

According to Darcy's law [Eq. (4.4)], the apparent velocities of flow of liquid through porous media are

$$u = -k\frac{\partial h}{\partial x}, \quad v = -k\frac{\partial h}{\partial y}, \quad \text{and} \quad w = -k\frac{\partial h}{\partial z} \qquad (9.42)$$

where, k is the coefficient of permeability, h is the hydraulic head causing the flow, and u, v,and w, respectively, are the x-, y-, and z-components of velocity.

Substituting these in the continuity equation,

$$\frac{\partial u}{\partial x} + \frac{\partial v}{\partial y} + \frac{\partial w}{\partial z} = 0 \qquad (9.43)$$

and assuming that k is a constant, one obtains

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2} = 0 \qquad (9.44)$$

This is the well-known Laplace equation which governs the flow of liquid through porous medium. This equation implicily assumes that,

The soil is homogeneous and isotropic,

The voids are completely filled with water,

No consolidation or expansion of the soil takes place,

The soil and water are incompressible, and

The flow obeys Darcy's law and is steady. For two-dimensional flow, Eq. (9.44) reduces to

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} = 0 \qquad (9.45)$$

Equation (9.45) can be solved by graphical, analytical, numerical or some other suitable method, such as analogue method.

Graphical Solution of Seepage Equation

A graphical solution of Laplace equation results in two families of curves which intersect at right angle and form a pattern of 'square' figures known as flownet (Fig. 9.21). One set of lines is called the streamlines or flow lines along which water can flow through a cross-section. The other set of lines comprise what are called equipotential lines which are lines of equal head or energy level. Any flownet must satisfy the following basic requirements:

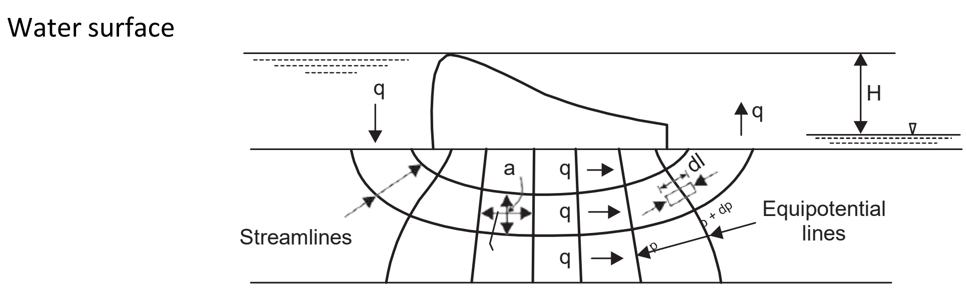


Fig. 9.21 Flownet for seepage under a weir

Flow lines and equipotential lines must intersect at right angle to form areas which are approximately squares. Most flownets are composed of curves and not straight lines. As such, the "square" figures are not true squares but curvilinear squares.The requirement of square figures is met, if the average width of any squire is equal to itsaverage length.

Certain entrance and exit requirements must be satisfied.

All pairs of adjacent equipotential lines must have equal head losses between them.

The same amout of seepage flows between all pairs of adjacent flow lines.

Using Darcy's law, a simple expression for seepage discharge can be obtained. Referring to Fig. 9.21, let the number of flow channels be N_f, the number of equipotential drops be N_d, and the seepage quantity flowing between any two adjacent flow lines be equal to Δq. Thus, the total seepage quantity,

$$q = N_f \, \Delta q$$

or
$$q = N_f \, k \frac{\Delta h}{\Delta l} a$$

or
$$q = N_f k \Delta h$$

as $\Delta l = a$, since the flownet is composed of squares.

Since $\Delta h = H/N_d$,

$$q = N_f \, k \frac{H}{N_d} = kH \frac{N_f}{N_d} \tag{9.46}$$

Equation (9.46) enables computation of the seepage quantity.The ratio N_f / N_d is called the shape factor. The seepage quantity is, therefore, the product of the coefficient of permeability, the net head, and the shape factor.

Considering an elementary cylindrical element of soil of cross-sectional area dA and length dl along any flow line, shown in Fig. 9.21, one can formulate the expression for the net seepage force dF in the direction of flow as

$$dF = p\,dA - (p + dp)\,dA$$

or
$$dF = -\,dp\,dA$$

Therefore, seepage force per unit volume of soil,

$$\frac{dF}{dA\,dl} = -\frac{dp}{dl} = -\rho g \frac{dH}{dl} \tag{9.47}$$

As the head H decreases in the direction of flow, dH/dl is negative and, hence, the seepage force is positive in the flow direction.

Obviously, at the exit end (Fig. 9.21), the seepage force is vertical and may cause the lifting of soil particles resulting in piping failure. Hence, to provide safety against piping failure, the seepage force at the exit end must be less than the submerged weight of the soil particles. At the critical condition, the two forces will just balance each other. Thus,

$$-\rho g \frac{dH}{dl} = (1 - n)(\rho_s - \rho)g \tag{9.48}$$

Here, n is the porosity of the soil and $(\rho_s - \rho)g$ is the submerged weight of unit volume of soil particles. Dividing Eq. (9.48) by ρg, one gets

$$-\frac{dH}{dl} = (1 - n)\left(\frac{\rho_s}{\rho} - 1\right) \tag{9.49}$$

or
$$-\frac{dH}{dl} = (1 - n)(G - 1) \tag{9.50}$$

where, G is the relative density of soil.

The quantity dH/dl represents the hydraulic gradient at the exit (or, simply, the exit gradient) which is negative. The value of dH/dl given by Eq. (9.50) is termed the critical gradient which should not be exceeded in order to prevent failure by piping.

Assuming G = 2.65 (true for most of the river sands) and n = 0.4, the value of the critical gradient is approximately 1.0. In practice, however, the actual gradient at the exit is kept around 1/4 to 1/6 depending on the safety requirements.

Method for Determination of Seepage Pressure

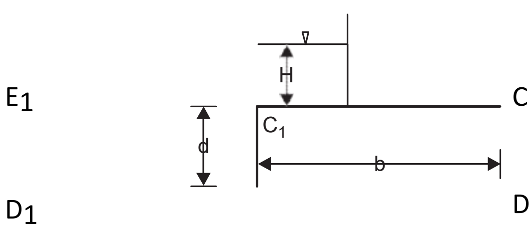
The seepage equation (or the Laplace equation), Eq. (9.45), cannot be solved exactly for usual canal structures having complex boundary conditions. Khosla, et al. (12) used the method of independent variables and obtained solutions of Laplace equation for a number of simple profiles. This solution is commonly known as Khosla's solution. The following forms of these simple profiles (Fig. 9.22) are very useful in the design of weirs and barrages on permeable foundations:

A straight horizontal floor of negligible thickness with a sheet pile at either end [Fig. 9.22 (i) and (ii)].

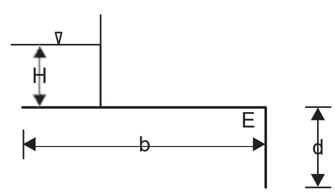
A straight horizontal floor of negligible thickness with an intermediate sheet pile [Fig. 9.22 (iii)].

A straight horizontal floor depressed below the bed but with no vertical cutoff [Fig. 9.22 (iv)]. This arrangement is useful for very small structures where no cutoff is provided.

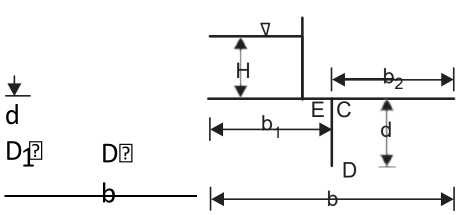
The solutions for these simple profiles have been obtained (12) in terms of the pressure head ratio ⍰ at 'key' points. These key points are the junction points of sheet piles with floor, i.e., E, C, D, E_1, C_1, and D_1 in case of floors of negligible thickness and D⍰ and D_1⍰ in case of depressed floor. The pressure head ratio ⍰ at any key point, say E, is thus H_E/H. Here, H_E is the uplift pressure head at E. The mathematical expressions for the pressure head ratio ⍰ at the key points are as follows:

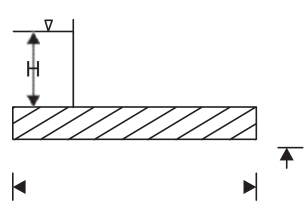


Sheet pile at the upstream end



Sheet pile at the downstream end





Intermediate sheet pile (iv) Depressed floor

Fig. 9.22 Simple standard profiles of weir floors.

For sheet piles at either the upstream end [Fig. 9.22 (i)] or the downstream end [Fig. 9.22 (ii)],

$$\phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right) \qquad (9.51)$$

$$\phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda - 1}{\lambda}\right) \qquad (9.52)$$

$$\phi_{C_1} = 100 - \phi_E \qquad (9.53)$$

$$\phi_{D_1} = 100 - \phi_D \qquad (9.54)$$

where, $\lambda = \frac{1}{2}\left[1 + \sqrt{1+\alpha^2}\,\right]$ (9.55)

and $\alpha = \frac{b}{d}$

For sheet piles at the intermediate point [Fig. 9.22 (iii)],

$$\phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1 - 1}{\lambda_2}\right) \qquad (9.56)$$

$$\phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1}{\lambda_2}\right) \qquad (9.57)$$

$$\phi_C = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1 + 1}{\lambda_2}\right) \qquad (9.58)$$

Here, $\lambda_1 = \frac{1}{2}\left[\sqrt{1+\alpha_1^2} - \sqrt{1+\alpha_2^2}\,\right]$ (9.59)

$\lambda_2 = \frac{1}{2}\left[\sqrt{1+\alpha_1^2} + \sqrt{1+\alpha_2^2}\,\right]$ (9.60)

where, $\alpha_1 = \frac{b_1}{d}$ and $\alpha_2 = \frac{b_2}{d}$

In the case of a depressed floor [Fig. 9.22 (iv)],

$$\phi_{D'} = \phi_D + \frac{2}{3}\,\frac{(\phi_E - \phi_D)}{\sqrt[3]{\alpha^2}} \qquad (9.61)$$

$$\phi_{D_1'} = 100 - \phi_{D'} \qquad (9.62)$$

where, ϕ_D and ϕ_E are given by Eqs. (9.52) and (9.51), respectively, and $\alpha = \frac{b}{d}$.

The above solutions have also been presented in the form of curves of Fig. 9.23 which too can be used for determining the pressure head ratio ϕ at the key points. One can directly obtain, from Fig. 9.23, the values of ϕ_C, and ϕ_D (for $b_1/b > 0.5$) for known values of α (= b/d) and b_1/b of intermediate pile [Fig. 9.22 (iii)]. The value of ϕ_E for given values of α (=b/d) and b_1/b of

intermediate pile is obtained by subtracting the value of $\square_C$ (for $1 - b_1/b$ and given □) from 100. For example, $\square_E$ [for $b_1/b = 0.4$ and □ = 4] = 100 – $\square_C$ (for $b_1/b = 0.6$ and □ = 4) = 100 – 29.1 = 70.9. Likewise, to obtain $\square_D$ (for $b_1/b < 0.5$ and given □), subtract $\square_D$ (for $1 - b_1/b$ and given □) from 100. For example, $\square_D$ [for $b_1/b = 0.4$ and □ = 4] = 100 – $\square_D$ (for $b_1/b = 0.6$ and □ = 4) = 100 – 44.8
= 55.2.

Similarly, while ϕ_E and ϕ_D for the sheet pile at the downstream end (Fig. 9.22 (ii) can be read from Fig. 9.23, the values of ϕ_C and ϕ_D for the sheet pile at the upstream end (Fig. 9.22 (i)) are obtained as follows:

$\phi_{C1} = 100 - \phi_E$ (for the sheet pile at the downstream end)

$\phi_{D1} = 100 - \phi_D$ (for the sheet pile at the downstream end) And for the depressed floor (Fig. 9.22 (iv)),

$\phi_{D1'} = 100 - \phi_{D'}$

The uplift pressures obtained for the simple forms from either the above-mentioned mathematical expressions or the curves of Fig. 9.23 are corrected for (i) the floor thickness, (ii) mutual interference of sheet piles, and (iii) the slope of the floor.

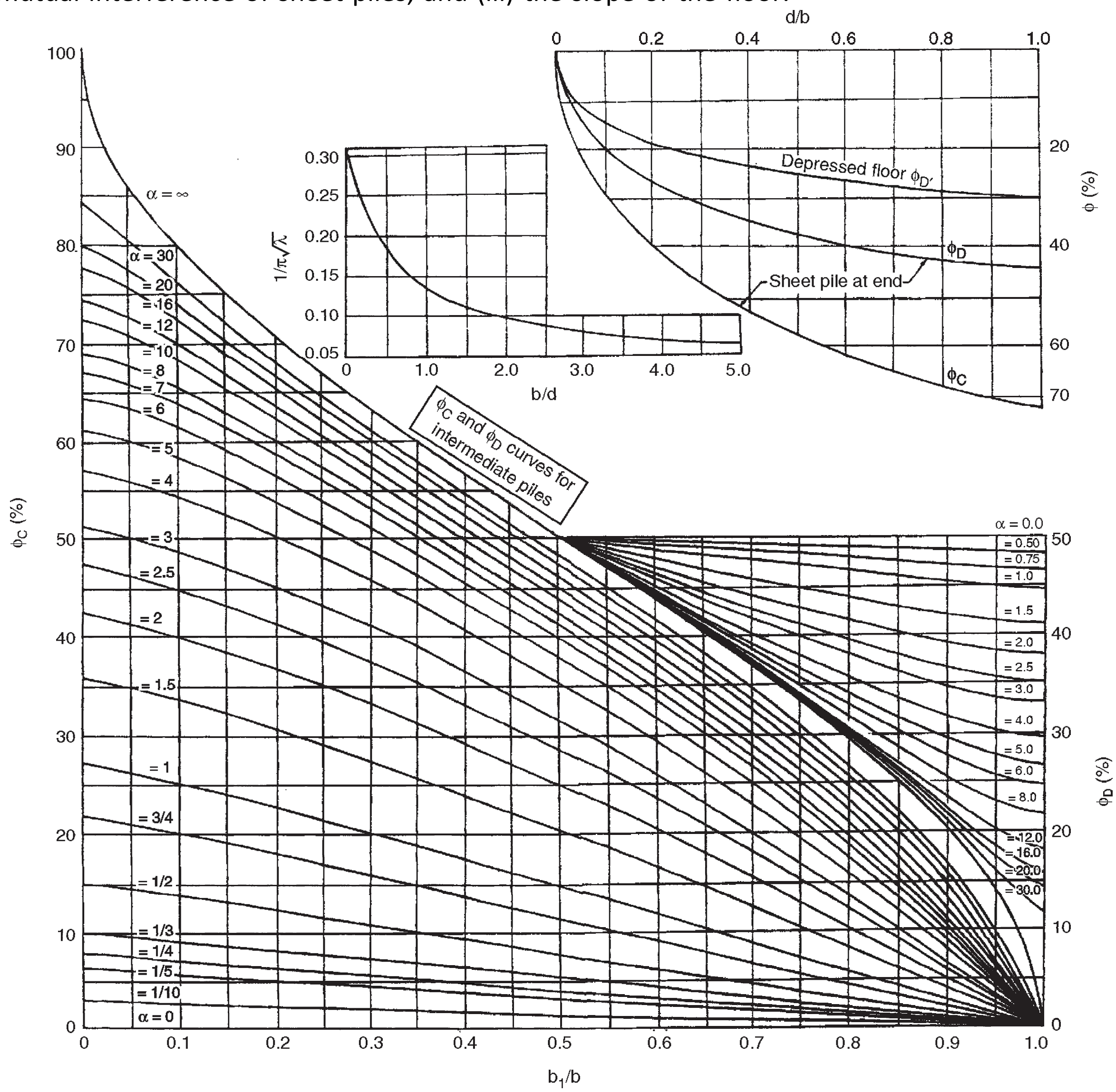


Fig. 9.23 Variation of ϕ and $\frac{1}{\pi\sqrt{\lambda}}$ (12)

The corrected values of the pressures are valid for actual profiles. The pressures at intermediate points between the adjacent key points are assumed to vary linearly. This assumption causes only negligible error.

Correction for Floor Thickness

The key points E (or E_1) and C (or C_1) correspond to the level at the top of the floor. The values of pressure at points E' (or E_1') and C' (or C_1') (Fig. 9.24) are interpolated assuming linear variation of pressure between the key points. Thus,

$$\phi_{E'} = \phi_E - \frac{\phi_E - \phi_D}{d - t}\, t \qquad (9.63)$$

$$\phi_{C'} = \phi_C + \frac{\phi_D - \phi_C}{d - t}\, t \qquad (9.64)$$

Here, t is the thickness of the floor.

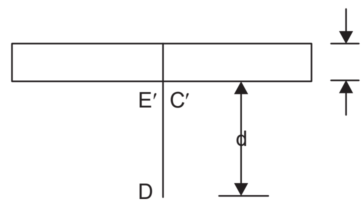


Fig. 9.24 Sketch for correction for floor thickness

Correction for Mutual Interference of Sheet Piles

Referring to Fig. 9.25, the amount of correction C (in per cent) for interference of sheet pile B for the key point C_{1a} of pile A is given as

$$C_i = 19 \sqrt{d_1/b'}\, \frac{d_1 - d}{b} \qquad (9.65)$$

where, b' is the distance between the two pile lines, d_1 the depth of the interfering pile (i.e., the pile whose influence is to be determined on the neighbouring pile of depth d) measured below the level at which the interference is desired, and b is the total floor length.

The correction C_i is additive for upstream (in relation to the interfering pile) points and negative for points downstream of the interfering pile. Equation (9.65) is not applicable for determining the effect of an outer pile on an intermediate pile when the latter is equal to or smaller than the former and is at a distance less than or equal to twice the length of the outer pile. The correction for interference of a pile is calculated only for the key points of the adjacent pile towards the interfering pile. In Fig. 9.25, pile B interferes with the downstream face (i.e., C_{1a}) of pile A and the upstream face (i.e., E_c) of pile C.

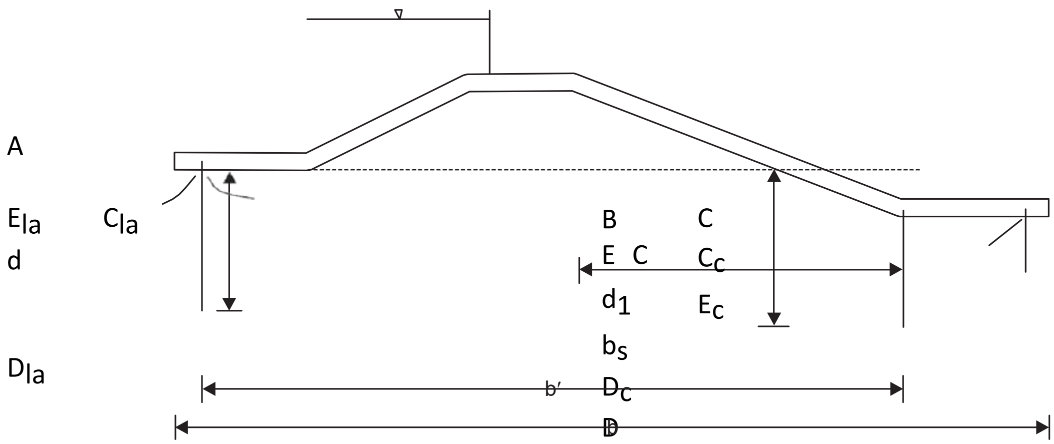


Fig. 9.25 Sketch for mutual interference of sheet piles and for the slope of the floor

Correction for the Slope of the Floor

The correction for the slope of the floor is applied to the pressures of the key point (on the side of the sloping floor) of that pile line which is fixed at either the beginning or the end of the slope. The correction is additive for positive slope (i.e., level of the floor is decreasing in the direction of flow) and is negative for the negative slope. In Fig. 9.25, the correction for slope is applicable only to the pressure at E of pile B and is positive. If b_s is the horizontal length of the sloping floor and b' is the distance between the two pile lines between which the sloping floor is located, then the amount of slope correction is equal to $C_s(b_s/b')$. The value of C_s depends on the slope of the sloping floor and is as given in Table 9.2.

Table 9.2 Values of C_s

Slope (V : H)	1 : 1	1 : 2	1 : 3	1 : 4	1 : 5	1 : 6	1 : 7	1 : 8
Correction C_s (% of pressure)	11.2	6.5	4.5	3.3	2.8	2.5	2.3	2.0

Method for Determination of Exit Gradient

For the simple profile of the downstream sheet pile, [Fig. 9.22 (ii)], the exit gradient G_E, as obtained by Khosla, et al. (12), is given as

$$G_E = \frac{H}{d}\,\frac{1}{\pi\sqrt{\lambda}} \qquad (9.66)$$

Equation (9.66) gives G_E equal to infinity for no sheet pile at the downstream end of the floor (i.e., d = 0). It is, therefore, necessary that a vertical cutoff (i.e., sheet pile) is always provided at the downstream end of the floor. To prevent piping, the exit gradient must not be allowed to exceed the critical value of the exit gradient which depends on the type of soil. The value of the critical exit gradient for sand varies from 1/5 to 1/7. One can also obtain the values of $1/\pi\sqrt{\lambda}$ from Fig. 9.23 and thus obtain the exit gradient.

Example 9.2 Using Khosla's method, obtain the residual seepage pressures at the 'key' points for the weir profile shown in Fig. 9.26. Also calculate the value of the exit gradient. Consider the case of no flow at pond level.

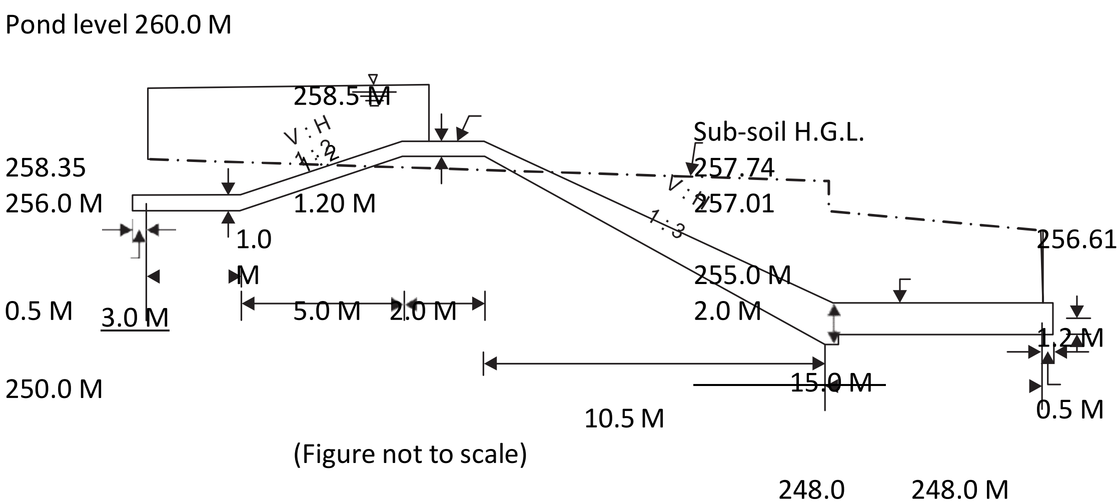


Fig. 9.26 Figure for Example 9.2

Solution: Upstream Pile :

Depth of pile $d = 256.00 - 250.00 = 6.0$ m Total floor length $b = 36.5$ m

Using Eqs. (9.51) to (9.55),

$$\alpha = \frac{b}{d} = \frac{36.5}{6.0} = 6.083$$

$$\lambda = \frac{1}{2}\left[1 + \sqrt{1+\alpha^2}\right] = 3.083$$

$$\phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-2}{\lambda}\right) = 0.386$$

$$\therefore \quad \phi_{C_1} = 1 - 0.386 = 0.614 = 61.4\%$$

$$\phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-1}{\lambda}\right) = 0.264$$

$$\therefore \quad \phi_{D_1} = 1 - 0.264 = 0.736 = 73.6\%$$

Using Fig. 9.23 with $\frac{1}{\alpha} = \frac{d}{b} = \frac{6.0}{36.5} = 0.164$

$\phi_{C_1} = 100 - \phi_E = 100 - 36 = 64\%$

and $\phi_{D_1} = 100 - \phi_D = 100 - 25 = 75\%$

It may be noted that there is slight difference between the values calculated from mathematical expressions and those read from Fig. 9.23. Corrections have been calculated and applied to the values obtained from mathematical expressions.

Thickness correction for $\phi_{C_1} = \frac{73.6 - 61.4}{6.0} \times 1.0 = 2.03\%$ (+)

Correction for interference of intermediate pile on ϕ_{C_1} of upstream pile

$$= 19\sqrt{\frac{(255-248)}{20.5}}\,\frac{[(255-248)+5]}{36.5} = 3.65\%\ (-)$$

$\therefore \quad \phi_{C_1}$ (corrected) = 61.4 + 2.03 + 3.65 = 67.08%

Intermediate Pile:

Using Eqs. (9.56) to (9.60),

$$\alpha_1 = \frac{b_1}{d} = \frac{21.0}{7.0} = 3.0$$

$$\alpha_2 = \frac{b_2}{d} = \frac{15.5}{7.0} = 2.2143$$

$$\lambda_1 = \frac{\sqrt{1+\alpha_1^2} + \sqrt{1+\alpha_2^2}}{2} = 2.796$$

$$\lambda_2 = \frac{\sqrt{1+\alpha_1^2} - \sqrt{1+\alpha_2^2}}{2} = 0.366$$

$$\phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_2 - 1}{\lambda_1}\right) = \frac{1}{\pi}\cos^{-1}\left(\frac{-0.634}{2.796}\right) = 0.5728 = 57.28\%$$

$$\phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_2}{\lambda_1}\right) = \frac{1}{\pi}\cos^{-1}\left(\frac{0.366}{2.796}\right) = 0.4582 = 45.82\%$$

$$\phi_C = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_2 + 1}{\lambda_1}\right) = \frac{1}{\pi}\cos^{-1}\left(\frac{1.366}{2.796}\right) = 0.3375 = 33.75\%$$

Using Fig. 9.23 with

b = 36.5 m, b_1 = 21.0 m, d = 255 – 248 = 7.0 m, and b_2 = 15.5 m

$$b_1/b = \frac{21.0}{36.5} = 0.575$$

$\therefore \quad 1 - (b_1/b) = 0.425$

and $\quad \alpha = \frac{36.5}{7.0} = 5.21$

$\therefore \quad \phi_C = 34\%$

$\phi_E = 100 - 42 = 58\%$

and $\quad \phi_D = 46\%$

The values of ϕ obtained from Eqs. (9.56) to (9.60) have been corrected as follows:

Thickness correction for ϕ_E $= \frac{57.28 - 45.82}{7} \times 2$ = 3.28% (–)

Slope correction for ϕ_E $= C_s\,(b_s/b') = 4.5 \times \left(\frac{10.5}{20.5}\right)$ = 2.31% (+)

Correction for interference of the upstream pile (for ϕ_E)

= 19

= 1.59% (–)

$$\sqrt{\frac{(253 \,\square\, 250)}{20.5}} \quad \frac{[(253 \,\square\, 250) \,\square\, 5]}{36.5}$$

$\square_E$(corrected) = 57.28 – 3.28 + 2.31 – 1.59 = 54.72%

Thickness correction for ϕ_C $= \frac{45.82 - 33.75}{7} \times 2 = 3.45\%\ (+)$

Correction for interference of the downstream pile (for ϕ_C)

$= 19\sqrt{\frac{(253-248)}{15}} \times \frac{(253-248)+5}{36.5} = 3.01\%\ (+)$

$\therefore$ ϕ_C (corrected) = 33.75 + 3.45 + 3.01 = 40.21%

Downstream pile:

d = 255 – 248 = 7.0 m and b = 36.5 m

$\therefore \alpha = \frac{36.5}{7.0} = 5.21$

Using Eqs. (9.55), (9.51) and (9.52),

$$\lambda = \frac{1}{2}\left[1 + \sqrt{1+\alpha^2}\right] = \frac{1}{2}\left[1 + \sqrt{1+(5.21)^2}\right] = 3.15$$

$$\phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-2}{\lambda}\right) = \frac{1}{\pi}\cos^{-1}\left(\frac{1.15}{3.15}\right) = 0.3810 = 38.10\%$$

$$\phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-1}{\lambda}\right) = \frac{1}{\pi}\cos^{-1}\left(\frac{2.15}{3.15}\right) = 0.2609 = 26.09\%$$

Using Fig. 9.23 with $\frac{1}{\alpha} = \frac{d}{b} = \frac{7.0}{36.5} = 0.192$,

ϕ_E = 38%

and ϕ_D = 26%

Using the values of ϕ_D and ϕ_E obtained from Eqs. (9.51) and (9.52),

Thickness correction for ϕ_E $= \frac{38.1 - 26.09}{7} \times 1.2 = 2.06\%\ (–)$

Correction for interference of intermediate pile (for ϕ_E)

$= 19\sqrt{\frac{(253.8-248)}{15}} \times \frac{(5.8+5.8)}{36.5}$

= 3.75% (–)

ϕ_E (corrected) = 38.1 – 2.06 – 3.75 = 32.29%

Based on the above calculations, subsoil hydraulic gradient line has been plotted in Fig. 9.26

Exit Gradient:

$\alpha = \frac{36.5}{7.0} = 5.21$; H = 260 – 255 = 5.0 m

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = 3.15$$

$$G_E = (H/d)\frac{1}{\pi\sqrt{\lambda}} = \frac{5.0}{7.0} \times \frac{1}{3.14\sqrt{3.15}} = 0.128 = 1 \text{ in } 7.8$$

UPLIFT FORCE ON THE FLOOR OF CANAL STRUCTURE

Seepage below the canal structure causes uplift force on the floor of the structure. This force is likely to be maximum when the water is ponded up to the highest level on the upstream side with no discharge on the downstream side. The subsoil hydraulic gradient line for this case has been shown in Fig. 9.27 and the net uplift pressure head at any section on the downstream

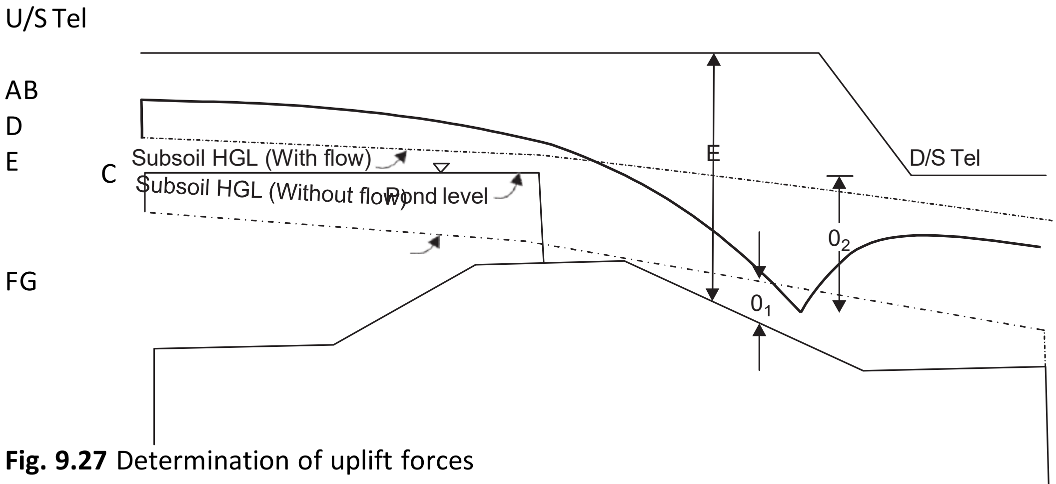
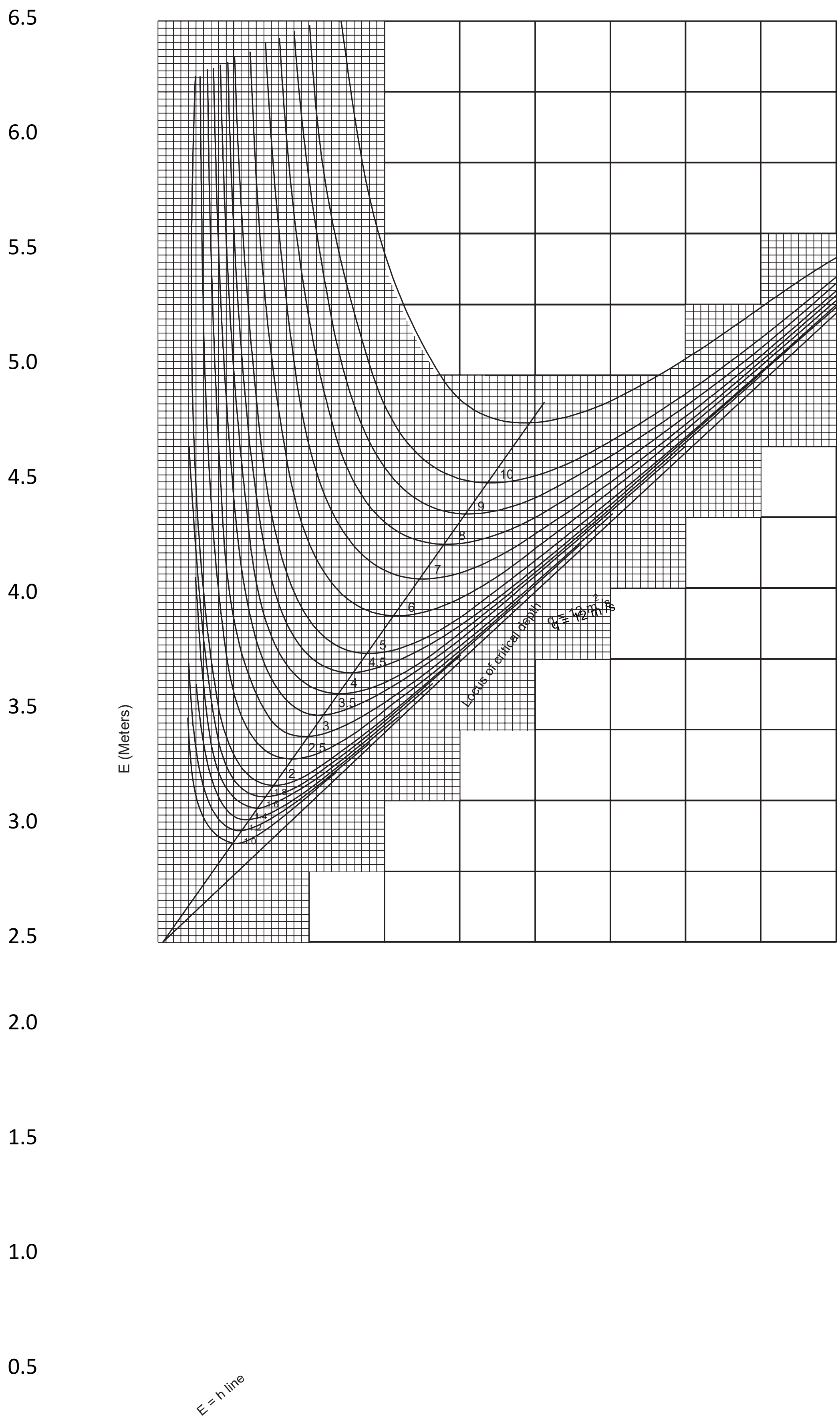


Fig. 9.27 Determination of uplift forces

of the barrier is the difference between the hydraulic gradient line and the floor level. This has been shown as o_1 in the figure. When there is flow, hydraulic jump forms and corresponding subsoil hydraulic gradient line for this case is also shown in Fig. 9.27. The net uplift pressure head for this condition will be obtained by measuring the distance of the water surface from the subsoil hydraulic gradient line at the desired section. This has been shown as o_2 in the figure. For most of the downstream sections, o_2 will be smaller than o_1. However, in the vicinity of the jump trough, o_2 may be greater than o_1. The floor thickness on the downstream side should, obviously, be based on the larger of the two values of o_1 and o_2.

For determining the uplift pressures at any section upstream of the jump trough the water surface profile needs to be determined. For this purpose, measure the difference between the levels of the total energy line and the floor at the section. This difference E is the value of specific energy at that section. Using Montague's curves (Fig. 9.28) or the specific energy equation, $E = h + (q^2/2gh^2)$, one can determine the supercritical depth of flow h for known E and q. In this way, one can obtain the water surface profile upstream of the jump. The jump profile can be obtained as explained in Sec. 9.2.4. The downstream sub-critical depth can alsobe obtained by solving the specific energy equation or using Montague's curves.



6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
E (Meters)
10
9
8
7
6
5
4.5
4
3.5
3
2.5
2
Locus of critical depth
q = 12 m²/s
E = h line

0

0	0.5	1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5	
		h								(Metres)

Fig. 9.28 Montague's curves

CANAL REGULATION STRUCTURES

GENERAL

Canal regulation structures are hydraulic structures which are constructed to regulate the discharge, flow velocity, or supply level in an irrigation channel. These structures are necessary for efficient working as well as for the safety of an irrigation channel. Canal regulation structurescan be classified as follows:

Canal fall: The canal fall (or, simply, the 'fall' or 'drop') regulates the supply level ina canal by negotiating the change in its bed elevation necessitated by the difference in ground slope and canal slope.

Distributary head regulator: This controls the supply to an offtaking channel from the parent channel.

Cross regulator: This structure controls the water level of a channel and the dis- charge downstream of another hydraulic structure.

Canal escape: Canal escape disposes of extra supplies when the safety of a canal is endangered due to heavy rains or closure of outlets by farmers.

CANAL FALL

A canal fall is a hydraulic structure constructed across a canal to lower its water level. This is achieved by negotiating the change in bed elevation of the canal necessitated by the difference in ground slope and canal slope. The necessity of a fall arises because the available ground slope usually exceeds the designed bed slope of a canal. Thus, an irrigation channel which is incutting in its head reach soon meets a condition when it has to be entirely in filling. An irrigation channel in embankment has the disadvantages of: (i) higher construction and maintenance cost, (ii) higher seepage and percolation losses, (iii) adjacent area being flooded due to any possible breach in the embankment, and (iv) difficulties in irrigation operations. Hence, an irrigation channel should not be located on high embankments. Falls are, therefore, introduced at appropriate places to lower the supply level of an irrigation channel. The canal water immediately downstream of the fall structure possesses excessive kinetic energy which, if notdissipated, may scour the bed and banks of the canal downstream of the fall. This would also endanger the safety of the fall structure. Therefore, a canal fall is always provided with measuresto dissipate surplus energy which, obviously, is the consequence of constructing the fall.

The location of a fall is primarily influenced by the topography of the area and the desirability of combining a fall with other masonry structures such as bridges, regulators, and so on. In case of main canals, economy in the cost of excavation is to be considered. Besides, the relative economy of providing a large number of smaller falls (achieving balanced earth work and ease in construction) compared to that of a smaller number of larger falls (resulting in reduced construction cost and increased power production) is also worked out. In case of channels which irrigate the command area directly, a fall should be provided before the bed of the channel comes into filling. The full supply level of a channel can be kept below the ground level for a distance of up to about 500 metres downstream of the fall as the command area in this reach can be irrigated by the channels offtaking from upstream of the fall.

HISTORICAL DEVELOPMENT OF FALLS

There was no theory or established practice for the design and construction of falls in the nineteenth century. Falls were usually avoided by providing sinuous curves in the canal alignment. This alternative increased the length of the canal. Obviously, this approach was uneconomical and resulted in an inefficient irrigation system.

The ogee fall (Fig. 10.1) was first constructed by Cautley on the Upper Ganga canal with a view to providing a smooth transition between the upstream and the downstream bed levels so that flow disturbances could be reduced as far as practicable. The smooth transition of the ogee fall preserved the kinetic energy and also resulted in large drawdown which caused heavy erosion of bed and banks on the downstream as well as upstream of the fall. Later, this type of fall was converted into a vertical impact type so as to cause more energy dissipation downstream of the fall.

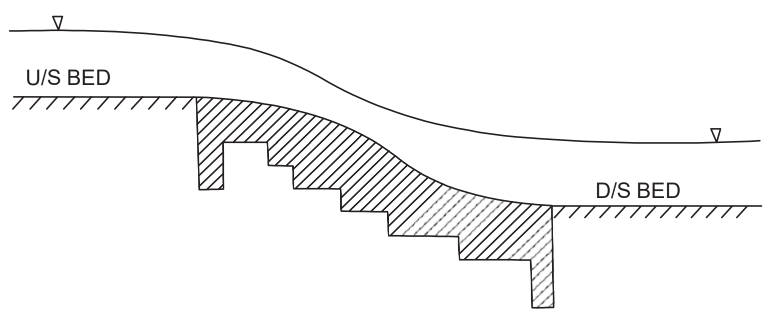


Fig. 10.1 Ogee fall

The falls on the Western Yamuna canal were in the form of 'rapids' (Fig. 10.2) which were gently sloping floors constructed at a slope of about 1 in 10 to 1 in 20.These rapids worked satisfactorily and permitted the movement of timber logs also. But, these structures were veryexpensive due to their longer length.

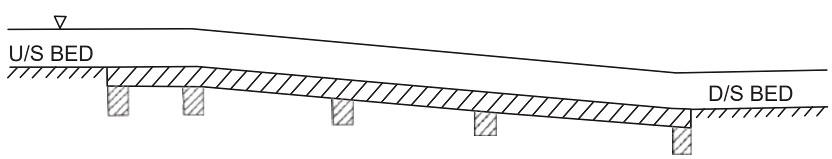


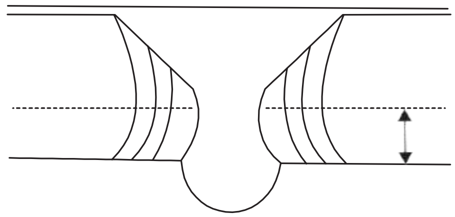
Fig. 10.2 Rapids

The realisation of the importance of a raised crest wall at the location of fall in reducing the drawdown resulted in the design of trapezoidal notch fall (Fig. 10.3). The fall structure had a number of trapezoidal notches in a high breast wall constructed across the channel having smooth entrance and a flat lip projecting downstream to spread out the falling jet. Such falls were very popular till simpler and economical falls were developed.

Plane of notch profile

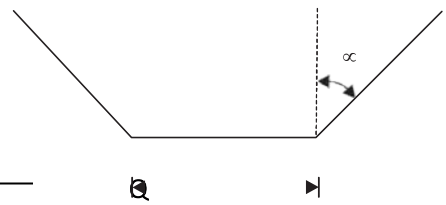
Setback

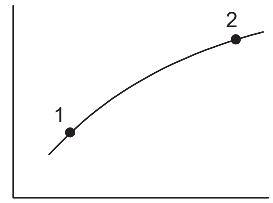
Lip



Plan
Trapezoidal notch fall

H





L

Dimensions of trapezoidal notch fall (c) Depth-discharge relation

Fig. 10.3 Trapezoidal notch fall

After World War I, the Sarda and glacis falls were developed in UP and Punjab, respectively. Of these two, the former proved to be more successful. The glacis fall gave trouble because of the resulting increase in discharge per unit width on account of fluming and, hence, the increased amount of kinetic energy to be dissipated downstream of the fall.

TYPES OF CANAL FALL

Canal falls are generally one of the following types:

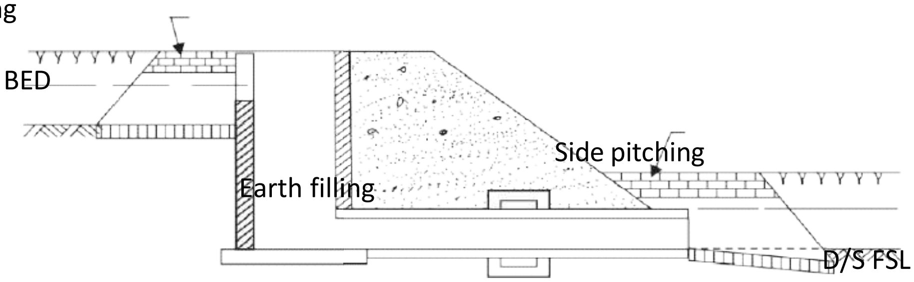
Canal falls which nearly maintain the normal depth-discharge relationship: Notch falls—trapezoidal or rectangular in shape—are of this type. The rectangular notch or low weir is not able to maintain accurately the normal depth-discharge relation- ship, but is economical and more suitable for discharge measurement. In a trapezoidal notch fall, a number of trapezoidal notches are made in a high breast wall across the channel. This arrangement provides an opening for flow right up to the bed level and thus eliminates silting in the channel upstream of the fall. The shape of the trapezoidal notch is decided on the basis of full supply and half supply conditions (1).

Canal falls which nearly maintain a fixed water surface level in the upstream chan- nel: When either a subsidiary channel takes off upstream of a fall, or the fall is combined with a hydro-electric plant, it is desirable that the water surface in the parent channel be maintained at a fixed level as far as possible. Siphon falls and high-crested weir falls fulfil this requirement. However, siphon falls, although very efficient, are too expensive and, hence, used only as siphon spillways in dams and not used as canal falls. High-crested weir falls are usually not flumed so as to keep the discharge per metre length of fall, q, small. The smaller the discharge intensity, the smaller is the head required and, hence, the water level upstream of the fall can be maintained at a relatively fixed level to a considerable extent. A smaller value of q also makes energy dissipation easier. Such falls are, therefore, relatively cheaper.

Generally, the length of a fall is limited to the width of the channel but, can be increased by providing an expansion followed by contraction in the channel. How- ever, this type of provision would increase the construction cost of the fall. Depend- ing upon the type of the weir crest, whether broad or narrow, and the flow condition, whether free or submerged, one can use these falls as metering devices after suitable calibration.
A raised-crest fall with vertical impact was first used on the Sarda canal system in UP. The amount of the drop at any fall in this canal system does not exceed 1.80 m. A large number of smaller falls were necessitated on this system because of the stratum of pure sand lying below the thin stratum of clay sand. Therefore, the depth of excavation for channel construction had to be kept low so as to keep the seepage losses to the minimum.
Canal falls which permit variation of water level upstream of the fall: The necessity of such falls arise when the subsidiary channel upstream of the fall has to be fed with minimum supply level in the parent channel. Such falls consist of either trapezoidal or rectangular notches. But, trapezoidal notches are relatively expensive and render the operation of regulators, such as stop logs and vertical strips, more difficult. In general, falls of this category, therefore, consist of rectangular notches combined with one of the following three types of regulators:
Sluice gate—Raising or lowering the gate helps in controlling the upstream level.
Horizontal stop logs inserted into grooves—Their removal or insertion causes the required change in the upstream level.
Vertical strips (or needles)—These change the effective width (i.e., the width of opening) of the channel and do not cause silting.
In addition to the above main types of falls, there may be falls designed to meet specific requirement. Cylindrical falls (also known as well falls), pipe falls, chutes, etc. are falls designed for specific requirements. For example, a cylindrical fall (Fig. 10.4) is suitable for low discharge and high drop whereas chutes or rapids may have to be constructed if the canal is to carry timber logs as well.

Side pitching

U/S FSLU/S BED



Slope 1 in 10

Fig. 10.4 Cylindrical fall
Canal falls can, alternatively, be divided on the basis of their capability to measure discharge. Accordingly, they may be either meter falls or non-meter falls.
CISTERN ELEMENT
As a result of the flow passing over a fall, the potential energy of the flow gets converted into kinetic energy. This excess kinetic energy, if not dissipated properly, will result in undesirable

scour of the bed and sides of the downstream channel. Hence, provision of means to dissipate the surplus kinetic energy is essential in all types of canal falls and is provided in a portion of the canal fall known as cistern element.

The cistern element or, simply, cistern, Fig. 10.5, located at the downstream of the crest of a fall structure, forms an important part of any canal fall. The cistern element is defined as that portion of the fall structure in which the surplus energy of the water leaving the crest is dissipated and the subsequent turmoil stilled, before the water passes into the lower level channel (1). The cistern element includes glacis, if any, devices for ensuring the formation of hydraulic jump and deflecting the residual high velocity jets, roughening devices, and the pool of water in which hydraulic impact takes place. In other words, the cistern element includes the complete structure from the downstream end of the crest to the upstream end of the lower channel.

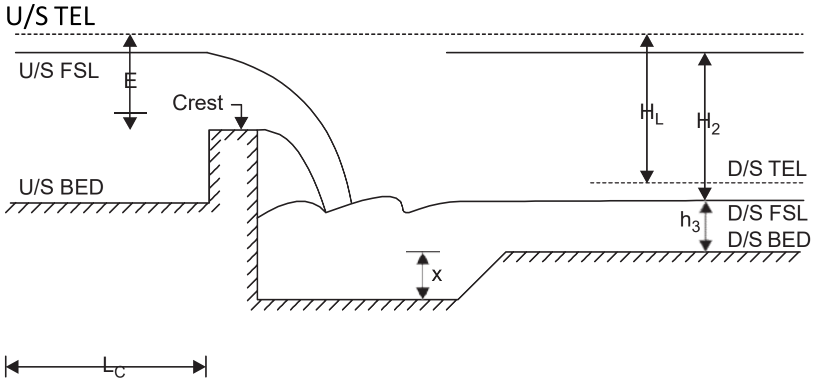


Fig. 10.5 Cistern element (vertical impact cistern)

Most of the cisterns employ hydraulic impact of the supercritical stream of the falling water with the subcritical stream of the lower channel for the dissipation of surplus energy. Depending upon the type of impact, the cisterns are divided into four categories discussed inthe following paragraphs.

Vertical-impact Cisterns

In these cisterns (Fig. 10.5) there is an impact of a stream of water falling freely. The path of such a stream is, obviously, parabolic. This type of cistern is very efficient for the dissipation ofsurplus energy when the drop is sufficient so that the falling stream becomes almost vertical. The dimensions of the cistern should be such that it serves the purpose of stilling and combingout the residual eddies and disturbances. The length L_C and depth x of the cistern can be determined only by empirical expressions given by different investigators. Some of these areas follows (1):

Dyas' formula	$x = \sqrt{H_L h_3}^{1/3}$	(10.1)
Glass' formula	$x + h_3 = 1.85\, E^{1/2} H_L^{1/3}$	(10.2)
	$L_C = 5\,(x + h_3)$	(10.3)
Etchevery formula	$L_C = 3\sqrt{E H_L}$	(10.4)
$x =$	$\frac{L_c}{6}$	(10.5)

UPIRI formula $x = \frac{1}{4}(EH_L)^{2/3}$ (10.6)

$L_C = 5\sqrt{EH_L}$ (10.7)

The symbols used in the above relations have been explained in Fig. 10.5 and theirvalues are in metres.

In most of the vertical-impact cisterns, roughening is not provided. Instead, a short length of cistern is provided for stilling and such a provision has been found to be satisfactory. To prevent the falling nappe from adhering to the masonry face of the fall, aeration of the nappe is necessary and is provided by aeration pipes embedded in the wing walls just downstream of the crest. The exit from the cistern should be smooth so that the flow is streamlined before it enters the downstream channel.

Considerations such as combining the fall with a bridge may require contracting the width of the channel at the site of the fall. This increases the discharge per unit width and alsothe surplus energy to be dissipated. The design of an efficient cistern may then be difficult and expensive. In such situations, one may have to design some other type of cistern even if the available drop is, otherwise, large enough and suitable for a vertical-impact cistern.

Horizontal-impact Cisterns

In this type of cistern (Fig. 10.6), water, after passing over the crest, flows on a glacis whose reverse curve at the downstream end turns the inclined supercritical flow to horizontal supercritical flow before it strikes the subcritical flow of the downstream channel resulting in the formation of a hydraulic jump. However, the position of a hydraulic jump on a horizontal floor is very sensitive to the variations in the depth or velocity of the downstream flow. As a result, surging (movement of the jump) is very likely to occur on the horizontal floor of this type of cistern. In the absence of a perfect jump, the energy dissipation may not be proper. Therefore, it is usual to depress the cistern downstream of the jump location until the depth in the cistern increases by about 25 per cent of the tail-water depth (1). Although this provision is generally adequate for energy dissipation, it should, however, be noted that when the jump forms on sloping floor, the impact is no longer horizontal. Inglis has, instead, suggested a low baffle for holding the jump on a horizontal floor (1).

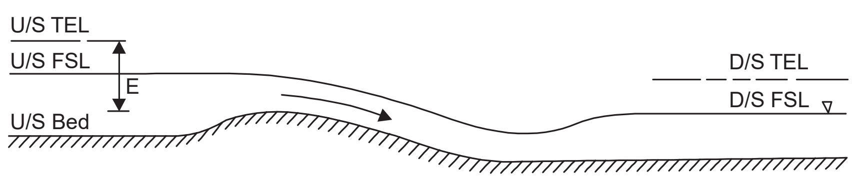


Fig. 10.6 Horizontal-impact cistern

For known discharge intensity q and the drop H_L, the total energy downstream of the jump E_2 can be calculated. The level of the cistern bed is then fixed at 1.25 E_2 below the downstream total energy line. Thus, the level of the bed of the cistern is independent of the level of the downstream channel. The length of the cistern is usually kept equal to 5 to 6 times E_2 to keep the jump within the cistern. Roughening devices, if required, may be provided starting from a section which is at a distance of half the height of the jump from the toe of the jump.

The design for such a cistern becomes more complicated when the supercritical jet is to be splayed. In such situations, the discharge intensity will be greater in the central part of the channel resulting in a 'bowed' jump. The expansion, therefore, must be very gradual. Although the horizontal-impact is an efficient energy dissipator (so long as the hydraulic jump continues to form), this type of provision requires expensive devices to hold the jump. Hence, provision of this type of cistern is usually made for dam spillways rather than for canal falls.

Inclined-impact Cisterns

For such cisterns, the glacis is carried straight down into the cistern and reliance is placed upon the effectiveness of the jump forming on the glacis for dissipation of surplus energy. However, the vertical component of the supercritical jet is not affected by the impact and, hence, energy dissipation is inefficient.

The dimensions of such cisterns are also decided in the same way as in the case ofhorizontal-impact cisterns. But, because of imperfect energy dissipation, the cistern has to accommodate the roughening devices and, hence, the cistern length for this class of cisterns is more than that for the preceding classes of cisterns.

No-impact Cisterns

Hydraulic impact is possible only when a supercritical flow meets a subcritical flow. In low falls and falls with large submergence, hydraulic impact may not be possible. Hence, other means of energy dissipation are adopted. A properly designed baffle wall along with suitable roughening devices are useful means of energy dissipation for such cases. The depth of no-impact cisterns cannot be calculated from theoretical considerations. However, it is useful to depress the cistern floor below the bed of the downstream channel as much as is economically feasible (1). This results in a larger cistern volume and, hence, permits retention of water for a longer period. Besides, it necessitates the construction of an upward slope (at the exit of the cistern) which helps in suppressing large-scale turbulence.

ROUGHENING MEASURES FOR ENERGY DISSIPATION

Hydraulic impact is the best means of energy dissipation in most hydraulic structures including canal falls. Of the three possible types of impact cisterns, viz., vertical-impact, horizontal-impact, and inclined-impact cisterns, the vertical-impact cistern is the most effective while the inclined-impact cistern is the least effective. However, even in the case of an efficient vertical-impact cistern, the high turbulence persists downstream of the cistern and means for dissipatingthe residual energy are essential. In case of cisterns with no hydraulic impact, the roughening devices (provided on the cistern floor) are the only means to dissipate the surplus kinetic energy. Artificial roughness increases the actual wetted area which, in turn, increases the boundary friction. Besides, if correctly shaped and placed, the roughness increases the internal friction by increasing the interaction between high speed layers of the stream.

Provision of grids in a canal is impossible because of the presence of debris in the stream. As such, roughening has to be in the form of projections from the bed and sides of the channels. Projections are the most effective means of energy dissipation. The following roughening devicesare generally used.

Friction Blocks

Rectangular concrete blocks properly anchored into the cistern floor and projecting up to one-fourth the full supply depth are simple, effective and commonly used devices for dissipating surplus kinetic energy in hydraulic structures. The spacing between the blocks in a row is kept about twice the height of the blocks. Depending upon the need, two or more staggered rows of these friction blocks may be provided (Fig. 10.7).

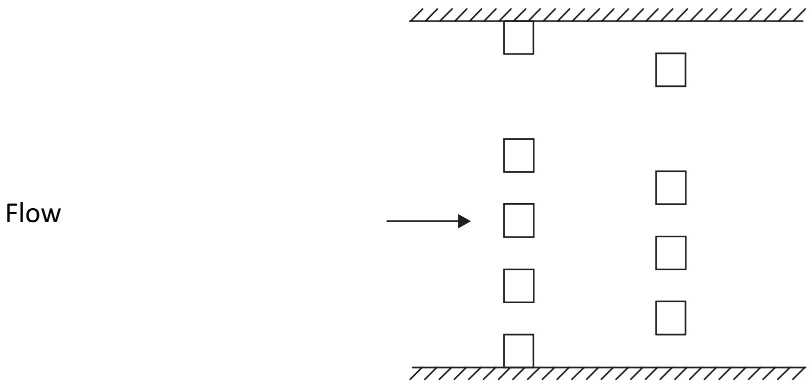


Fig. 10.7 Staggered friction blocks

The 'arrows' are specially shaped friction blocks (Fig. 10.8). The plan form of these arrows is approximately an equilateral triangle with rounded corners. The back face of arrows is vertical. The top of arrows is sloped from the front rounded corner to the back edge to give an upward deflection to stream filaments.

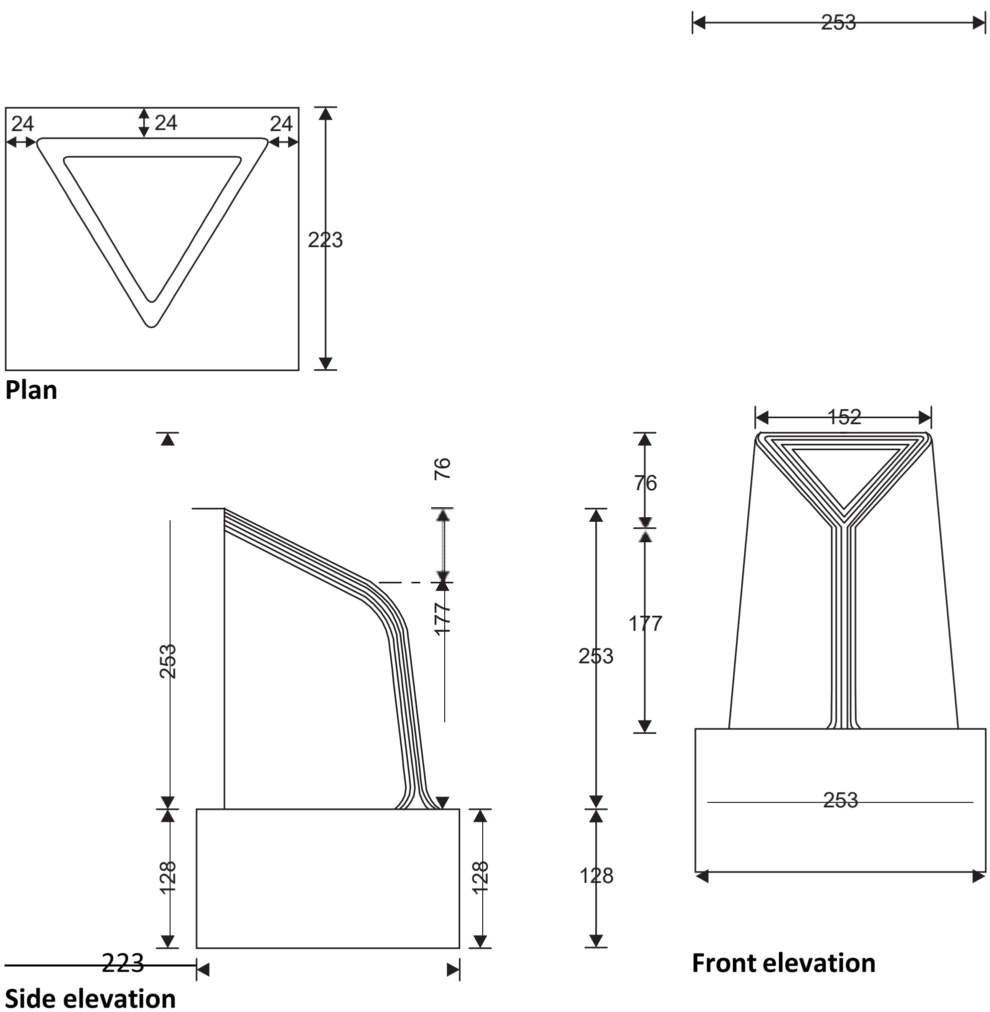


Fig. 10.8 Arrows

The length of cistern required to be roughened is equal to (1)

$$c \frac{h_3^{3/2} H_L^{1/2}}{D}$$

where, D is the depth of water in the cistern, h_3 the depth of flow in the downstream channel,
H_L the drop height and c is a coefficient whose value depends on the type of impact as follows: Vertical impact, c = 1
Horizontal impact, c = 3
Inclined impact with baffle, c = 4 Inclined impact without baffle, c = 6No impact, c = 8 to 10
Where hydraulic impact occurs, the roughening should start downstream of the jump at a distance about half the height of the hydraulic jump. Downstream of the roughened length, a smooth cistern (without roughening devices) about half as long as the roughened cistern should be provided (1).

Ribbed Pitching

Projections on the sides of the channel for the purpose of dissipating surplus energy of the flow can be provided in the form of ribbed pitching which consists of bricks laid flat and on edge alternately (Fig. 10.9). Bricks laid on edge project into the stream and thus increase boundary friction and dissipate the surplus energy.

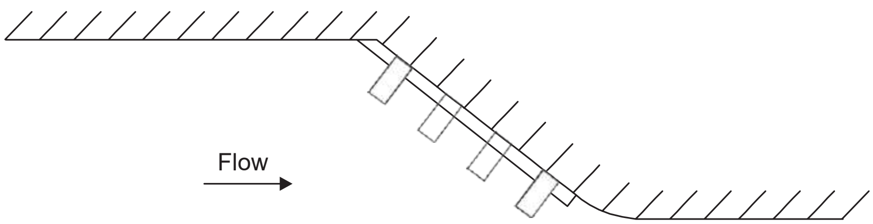
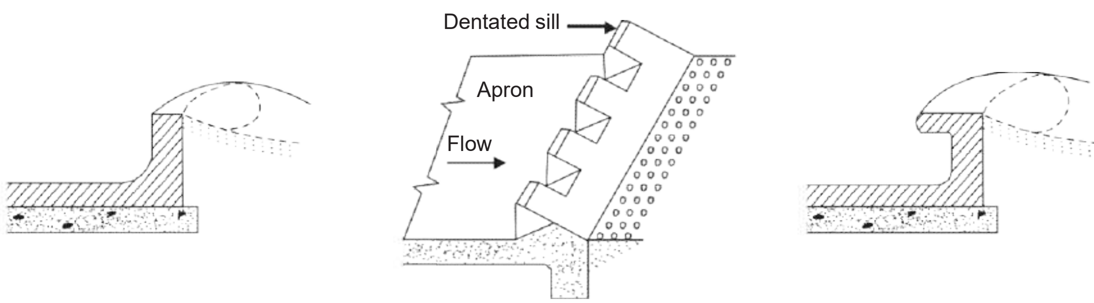


Fig. 10.9 Ribbed pitching

Provisions at the Downstream End of Cistern

If high velocity stream continues up to the end of the cistern, a baffle wall or a deflector [Fig. 10.10 (a)], or a dentated cill [Fig. 10.10 (b)] or a biff wall [Fig. 10.10 (c)] may be provided



Deflector Dentated sill Biff wall

Fig. 10.10 Roughening devices at the downstream of cistern at the downstream end of the cistern. The baffle wall provides a deep pool of water upstream of itself in the cistern. This pool of water is helpful in the dissipation of residual energy. Other devices (i.e., deflector, dentated cill, biff wall) produce a reverse roller which results in a limited scour away from the toe and piles up material against the toe of the structure. A dentated cill, in addition, breaks up the stream jet.

TRAPEZOIDAL NOTCH FALL

A trapezoidal notch fall can be designed [i.e., determine α and L, Fig. 10.3 (b)] to maintain the normal depth in the upstream channel for extreme values [say 1 and 2, Fig. 10.3 (c)] of a specified range of discharge, Q, using the following discharge equation for free flow condition:

$$Q = \frac{2}{3} C \sqrt{2g} \left[L H^{3/2} + \frac{4}{5} H^{5/2} \tan\alpha \right] \tag{10.8}$$

Here, H is the depth of water above the notch cill up to the normal water surface and is measured upstream of the fall where the streamlines are relatively straight. The value of the coefficient of discharge C may be taken as 0.78 for canal notches, and 0.70 for distributary notches (2). For two values of discharge, Q_1 and Q_2, and corresponding values of H_1 and H_2, one can obtain the following two equations for the determination of two unknowns L and α:

$$Q_1 = \frac{2}{3} C \sqrt{2g} \left[L H_1^{3/2} + \frac{4}{5} H_1^{5/2} \tan\alpha \right] \tag{10.9}$$

$$Q_2 = \frac{2}{3} C \sqrt{2g} \left[L H_2^{3/2} + \frac{4}{5} H_2^{5/2} \tan\alpha \right] \tag{10.10}$$

On solving Eqs. (10.9) and (10.10), one obtains

$$\tan\alpha = \frac{15}{8} \frac{Q_2 H_1^{3/2} - Q_1 H_2^{3/2}}{C \sqrt{2g}\, H_1^{3/2} H_2^{3/2} (H_2 - H_1)} \tag{10.11}$$

and

$$L = \frac{Q_1}{(2/3)\, C \sqrt{2g}\, H_1^{3/2}} - \frac{4}{5} H_1 \tan\alpha \tag{10.12}$$

Trapezoidal notch falls are designed for the full supply discharge and half of the full supply discharge (1).

Similarly, using the following equation for the submerged flow condition, the unknowns L and α can be determined for two sets of known values of Q, H and h_d (i.e., the submergence head) for the two stages of the channel (2):

$$Q = \frac{2}{3} C \sqrt{2g} (H - h_d)^{3/2} \{(L + 2h_d \tan\alpha) + 0.8 \tan\alpha (H - h_d)\} + C \sqrt{2g} (H - h_d)^{1/2} (L + h_d \tan\alpha)\, h_d \tag{10.13}$$

The number of notches is so adjusted that the top width of the flow in the notch lies between 3/4th to full water depth above the cill of the notch. The minimum thickness of notch piers is half the depth and can be more if the piers have to support a heavy superstructure.

SARDA FALL

It is a raised-crest fall with a vertical-impact cistern. For discharges of less than 14 m^3/s, a rectangular crest with both faces vertical is adopted. If the canal discharge exceeds 14 m^3/s, a trapezoidal crest with sloping downstream and upstream faces is selected. The slopes of the downstream and upstream faces are 1 in 8 and 1 in 3, respectively. Both types of crests (Fig. 10.11) have a narrow and flat top with rounded corners. In Sarda fall, the length of the crest L is generally kept the same as the channel width. However, for future and other specific requirements, the crest length may exceed the bed width of the channel by an amount equal tothe depth of flow in the channel.

For a rectangular-crest type Sarda fall, the discharge Q under free flow condition is expressed as (2)

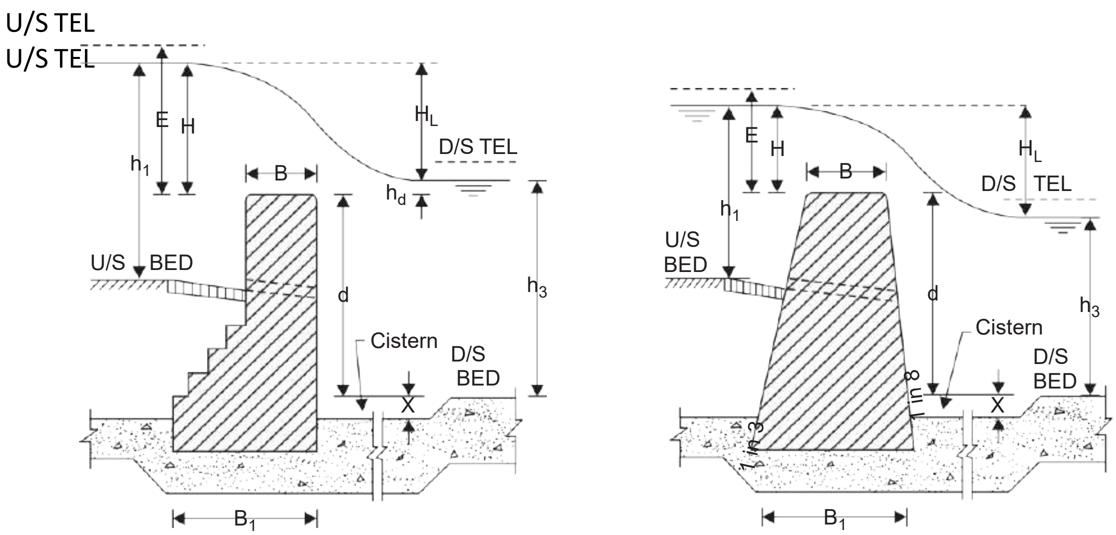
$$Q = 0.415\sqrt{2g}\,LH^{3/2}\left(\frac{H}{B}\right)^{1/6} \qquad (10.14)$$

Here, H is the head over the crest, and B is the width of crest which is related to the height of the crest above the downstream bed d as follows:

$$B = 0.55\sqrt{d} \qquad (10.15)$$

Obviously, $H + d = h_1 + D$

and, therefore, $d = h_1 + D - H$ (10.16)



(a) Rectangular crest (Drowned flow) (b)Trapezoidal crest (Free flow)

Fig. 10.11 Types of cross-sections of crest for Sarda fall

Here, D is the drop in the bed level, and h_1 is the upstream depth of flow. Equations (10.14) and (10.15) are solved by trial for obtaining the values of B and H for the rectangular-crest type Sarda fall.

For the trapezoidal-crest type Sarda fall, the discharge Q under free flow condition is expressed as (2)

$$Q = 0.45\sqrt{2g}\,LH^{3/2}\left(\frac{H}{B}\right)^{1/6} \qquad (10.17)$$

Here, $B = 0.55\sqrt{H+d}$

$$\therefore \quad B = 0.55\sqrt{h_1 + D} \qquad (10.18)$$

For known h_1 and D, one can determine B using Eq. (10.18). The head H is obtained from Eq. (10.17).

For submerged flow conditions, one should use the following discharge equation (2):

$$Q = C_d\sqrt{2g}\,L\left[\frac{2}{3}H_L^{3/2} + h_d\sqrt{H_L}\right] \qquad (10.19)$$

Here, H_L is the difference between the upstream and the downstream water levels, and h_d is the submergence head as shown in Fig. 10.11 (a). The coefficient of discharge C_d is usually assigned an average value of 0.65. Equation (10.19) ignores the effect of the approach velocity. For given conditions, one can determine h_d from Eq. (10.19) and, hence, the height of the crest above the upstream bed which equals $[h_1 - (H_L + h_d)]$. Alternatively, one can use the following equation for the submerged flow (3):

$$Q = C_d L\sqrt{2g}\left[\frac{2}{3}\{(H_L + h_a)^{3/2} - h_a^{3/2}\} + h_d\{H_L + h_a\}^{1/2}\right] \qquad (10.20)$$

Here, h_a is the approach velocity head.

The base width of a fall is decided by the requirement of concrete cover for stability and the slopes of the upstream and downstream faces of the fall.

A depressed cistern having suitable length and depression is provided immediately downstream of the crest and below the downstream bed level. The cistern and the downstream floor are usually lined with bricks laid on edge so that repair work would only involve brick surfacing and, hence, be relatively simple.

The depths of the cutoffs are as follows (3):

Depth of upstream cutoff $= \left(\frac{h_1}{3} + 0.6\right)$ m

with a minimum of 0.8 m

Depth of downstream cutoff $= \left(\frac{h_1}{2} + 0.6\right)$ m

with a minimum of 1.0 m

Here, h_1 is the upstream depth of flow in metres. The thickness of the cutoff is generally kept 40 cm.

The length of impervious floor is decided on the basis of either Bligh's theory or method proposed by Khosla et al. (Sec. 9.3.3). For all major works, the method of Khosla et al. should always be used. The most critical condition with respect to seepage would occur when water is up to the crest level with no overflow. The minimum floor length l_d, which must be provided downstream of a fall, is given by (2)

$$l_d = 2h_1 + H_L + 2.4$$

A thickness of 0.30 m is usually sufficient for the upstream floor. The thickness of the downstream floor is calculated from considerations of uplift pressure subject to a minimum of 0.45 to 0.60 m for larger falls and 0.3 to 0.45 m for smaller falls.

Brick pitching is provided on the channel bed immediately upstream of the fall structure. It is laid at a slope of 1 : 10 for a distance equal to the upstream depth of flow h_1. Few drain holes of diameter about 15 to 30 cm are provided in the raised-crest wall at the bed level to drain out the upstream bed when the channel is closed for maintenance or other purposes.

Radius of curvature of the upstream wing walls is around 5 to 6 times H. These walls subtend an angle of 60° at the centre and extend into earthen banks such that these are embedded in the channel banks by a minimum of 1 m. The wing walls should continue up to the end of the upstream impervious floor and for this purpose, if necessary, the walls may run along straight banks tangential to the wall segment.

Downstream of the fall structure, the wing walls are lowered down to the levels of the downstream wing walls through a series of steps. The downstream wing walls are kept vertical for a distance of about 5 to 8 times $\sqrt{EH_L}$ from the fall structure. These walls are then flared so that their slope changes from vertical to the side slope of the downstream channel. The wing walls are designed as the retaining walls to resist the earth pressures when the channel is not running.

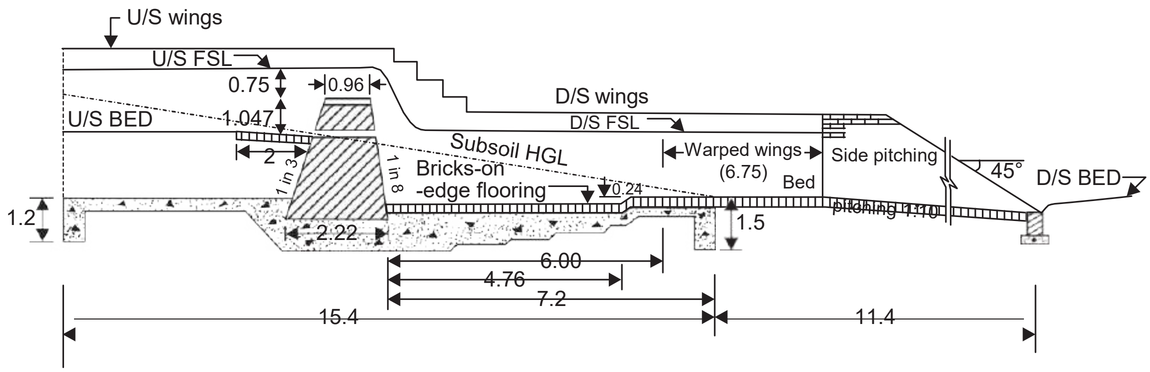
Downstream of the warped wing walls, pitching protection is provided on the bed and sides of the channel. Pitching is either brick work or stones laid dry on a surface without the use of mortar. Pitching is, therefore, pervious. It is provided for a length equal to about three times the downstream depth of flow (3). Alternatively, Table 10.1 can be used for deciding the length of bed pitching. Bed pitching is kept horizontal up to the end of wing walls and, thereafter, it slopes at 1 in 10. The pitching on the side slopes of the channel is provided up to the line originating from the end of the bed pitching and inclined at 45° towards the upstream (Fig. 10.12). A toe wall at the junction of the bed and the side pitching is necessary for providing firm support to the side pitching.

Table 10.1 Length of bed pitching (2)

Head over crest, H in metres	Total length of pitching in metres
Less than 0.30	3
0.30 to 0.45	3 + 2D
0.45 to 0.60	4.5 + 2D
0.60 to 0.75	6.0 + 2D
0.75 to 0.90	9.0 + 2D
0.90 to 1.05	13.5 + 2D
1.05 to 1.20	18.0 + 2D
1.20 to 1.50	22.5 + 2D
D = Drop in bed level.	

Example 10.1 Design a 1.2 m Sarda fall for a channel carrying 25 m^3/s of water at a depth of flow equal to 1.8 m. The bed width of the channel is 20 m.

Solution: Since the discharge exceeds 14 m^3/s, the cross-section of the crest of Sarda fall is chosen as trapezoidal with sloping downstream (1 in 8) and upstream (1 in 3) faces.



(All dimensions in metres)

Fig. 10.12 Longitudinal section of Sarda fall (Example 10.1) (not to scale)

Dimensions of crest: Using Eq. (10.18), Width of crest, $B = 0.55 \sqrt{h_1 + D}$

$= 0.55 \sqrt{1.8 + 1.2}$

$= 0.953$ m

$= 0.96$ m (say) Length of crest = Bed width of channel = 20.0 m From Eq. (10.17),

$$H = \left[\frac{Q B^{1/6}}{0.45 \sqrt{2g} L}\right]^{0.6}$$

$$= \left[\frac{25 \times (0.96)^{1/6}}{0.45 \times \sqrt{19.62} \times 20}\right]^{0.6} = 0.753 \text{ m}$$

Height of crest above the upstream bed $= h_1 - H$

$= 1.8 - 0.753$

$= 1.047$ m

Height of crest above the downstream bed, $d = h_1 + D - H$

$= 1.8 + 1.2 - 0.753$

$= 2.247$ m

The base of the fall should be at least 0.5 m below the downstream bed level. Accordingly,

Base width of fall $= (1/3)(d + 0.5) + B + (1/8)(d + 0.5)$

$= (11/24)(d + 0.5) + B$

$= (11/24)(2.247 + 0.5) + 0.96$

$= 2.220$ m

Cistern: Using Eq. (10.7),

Length of cistern, $L_C = 5 \sqrt{E H_L}$

Assuming $E \approx H$, $L_C = 5 \sqrt{0.753 \times 1.2}$

= 4.753 m
= 4.76 m (say)
From Eq. (10.6),

Depth of cistern, $x = (1/4)\,(EH_L)^{2/3}$

$= (1/4)\,(0.753 \times 1.2)^{2/3}$
= 0.234 m
= 0.24 m (say)

Upstream and downstream cutoffs:

Depth of the upstream cutoff = $\frac{h_1}{3} + 0.60$

$= (1.8/3) + 0.60$
= 1.2 m

Depth of the downstream cutoff = $\frac{h_1}{2} + 0.60$

$= (1.8/2) + 0.60$
= 1.5 m

Thickness of these cutoffs may be kept equal to 0.4 m.

Length of impervious floor: Assuming safe exit gradient to be equal to 1/5, one can write

$$G_E = \frac{1}{\pi}\left(\frac{H}{d}\right)\frac{1}{\sqrt{\lambda}}$$

Here, H = Head for no flow condition
= height of the crest above the downstream bed = 2.247 m
d = the depth of the downstream cutoff = 1.5 m

$$\therefore \quad \frac{1}{\pi\sqrt{\lambda}} = G_E \frac{d}{H} = \frac{1}{5} \times \frac{1.5}{2.247} = 0.134$$

$\therefore \quad \lambda = 5.643$

$\therefore \quad \alpha = \frac{b}{d_1} = 10.24$

$\therefore \quad b = 10 \times 1.5$

$\therefore \quad$ Total floor length = 15.36 m $\approx$ 15.4 m (say)

Minimum floor length required on the downstream = $2h_1 + H_L + 2.4$
$= 2 \times 1.8 + 1.2 + 2.4 = 7.2$ m

So provide the downstream floor length equal to 7.2 m and the balance 8.2 m long impervious floor on the upstream side. Thickness of the concrete floor at various sections is decided as illustrated in Example 10.2.

Upstream protection:

Radius of curvature of the upstream wing walls = 5 to 6 times H
= 5 × 0.753 to 6 × 0.753
= 3.765 m to 4.518 m
= 4.0 m (say)

Brick pitching on the upstream bed is provided at a slope of 1 : 10 for a distance equal to the upstream depth of flow. Drain holes of 20 cm diameter are provided at an interval of 4 m.

Downstream protection: The downstream wing walls are to be kept vertical for a distance of about 5 to 8 $\sqrt{EH_L}$

= 5 $\sqrt{0.753 \times 1.2}$ to 8 $\sqrt{0.753 \times 1.2}$

= 4.75 m to 7.60 m

= 6.0 m (say)

Thereafter, the wing walls should be warped from the vertical to the side slopes of the channel. The top of the wing walls is given an average splay of about 1 : 2.5 to 1 : 4. The difference in the surface widths of flow in the rectangular and trapezoidal section is 1.8 × 1.5 × 2 = 5.4 m. For providing a splay of 1 : 2.5 in the warped wings, the length of the warped wings along the channel axis is equal to (5.4/2) × 2.5 = 6.75 m.

In accordance with Table 10.1, total length of bed pitching on the downstream side is equal to 9 + 2D

= 9 + 2.4

= 11.4 m

The bed pitching should be horizontal up to the end of the downstream wing walls and, thereafter, it should be laid at a slope of 1 in 10. A toe wall of thickness equal to 40 cm and depth equal to 1.0 m should be provided at the end of the bed pitching.

Side pitching is provided downstream of the wing walls up to the end of bed pitching. The side pitching may be suitably curtailed. Longitudinal section based on these computations has been shown in Fig. 10.12.

GLACIS FALL

In case of a glacis fall, the energy is dissipated through the hydraulic jump which forms at the toe of the glacis. Ideally, the profile of the glacis should be such that the maximum horizontal acceleration is imparted to the falling stream of water in a given length of the structure to ensure maximum dissipation of energy. It is obvious that in case of a free fall under gravity, there will be only vertical acceleration and no horizontal acceleration. Theoretically speaking, there will be some horizontal acceleration on a horizontal floor at the crest level because of the formation of an H_2 profile. This acceleration, however, would be relatively small. Hence, there should be a glacis profile, in between the horizontal and vertical, which would result in the maximum horizontal acceleration. Neglecting the acceleration on a horizontal floor at the crest level, Montague obtained the following profile which would yield maximum horizontal acceleration (1):

x = 2U ⍰ y (10.21) $\sqrt{y/g}$

where, U is the initial horizontal velocity of water at the crest, and x and y are, respectively, the horizontal and vertical distances from the crest to any point along the glacis. The parabolic glacis profile [Eq. (10.21)] is, however, difficult and costly to construct. As such, a straightglacis with a slope of 2(H) : 1 (V) is commonly used.

A straight glacis fall may be provided with a baffle platform at the toe of the glacis and a baffle wall at the end of the platform in order to hold the jump at the toe of the glacis. The baffle platform is followed by a cistern downstream of the baffle wall. Such a fall was first developed by Inglis (1) and is called an Inglis fall; this too is obsolete now.

The present practice is to use a fall (Fig. 10.13) with straight glacis of slope 2 (H) : 1(V) at the toe of which is provided a cistern followed by friction blocks and other suitable measures to hold the jump at the toe of the glacis. Such straight glacis falls may or may not be flumed and can be either meter or non-meter falls. For known specific energy E at the downstream and the energy loss H_L in the jump, one can determine the discharge intensity q using Blench curves (Fig. 9.17). The crest length L is obtained by dividing the total canal discharge Q by the discharge intensity q. While finalising the crest length, the following limits with regard to permissible fluming should be kept in mind (3):

Drop	Permissible minimum value of the ratio ofcrest length to the width of canal
Up to 1 m	0.66
Between 1 and 3 m	0.75
Above 3 m	0.85

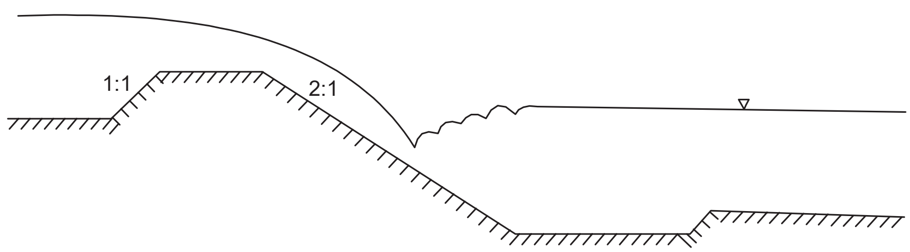


Fig. 10.13 Straight glacis fall

The level of the crest is H below the upstream total energy line. Here, H is the head over the crest up to the total energy line and is obtained by the discharge equation of a broad-crested weir,

$$Q = 1.71\ LH^{3/2} \qquad (10.22)$$

Here, Q is expressed in m^3/s, and L and H are in metres. For meter falls, the width of the crest should at least be 2.5 H so that the crest behaves as a broad-crested weir, and the coefficient of discharge is fairly constant at different discharges.

The upstream face of the crest is kept inclined at a slope of 1 : 1. The downstream face of the crest, i.e., the glacis, is kept straight and inclined at a slope of 2 (H) : 1(V). The glacis is usually carried below the bed level of the downstream channel so that the toe of the glacis (or the level of the cistern) is 1.25 E_2 below the downstream total energy line. The depressed cistern increases the post-jump depth by about 25 per cent of the tail-water depth. This increased depth ensures formation of the jump at the toe of the glacis. The length of the cistern should not be less than 2 E_2. The cistern joins the downstream concrete floor through an upward slope of 5 (H) : 1 (V). The length of the downstream concrete floor (inclusive of the cistern) should be about five to six times the height of the jump. A concrete floor is also provided in the upstream canal immediately upstream of the crest for a length of about 2.0 m. In addition, a vertical cutoff is also provided at the downstream end of the concrete floor. The length and thickness of the concrete floor are checked against uplift force and exit gradient.

In case of flumed falls, suitable expansion has to be provided starting from the downstream end of the glacis. The expansion can be either a hyperbolic type or, simply, a 1 : 3

straight expansion. The vertical side walls at the end of the cistern are so warped that at the end of the expansion, the slope of the side walls is equal to the side slopes of the downstream channel. It should be noted here that while deciding the width of the flumed portion, the presence of the downstream expansion was completely ignored. For the prevalent subcritical flow conditions, the tail-water depth at the upstream end of the expansion will be smaller than the tail-water depth in the downstream channel after the expansion. To ensure formation of the jump at the toe of the glacis, friction blocks and end cill (both of around 0.5 m height) are additionally provided on the concrete floor. These provisions along with the depressed cistern ensure formation of the jump at the toe of the glacis. These design specifications have stood the test of time and are, accordingly, used in practice.

Downstream of the concrete floor, brick pitching is provided on the bed up to the end of the expansion. Brick pitching is also provided on the upstream canal bed just upstream of the concrete floor. Toe walls are suitably provided to support the brick pitching.

For meter falls, the side walls immediately upstream of the crest are made curved with radius of curvature of five to six times the drop and subtending an angle of 60° at the centre. These curved wing walls are joined to the upstream canal banks. For non-meter falls, however, the side walls may only be splayed at an angle of 45° from the upstream edge of the crest and carried into the banks for about 1.0 m.

Example 10.2 Design a straight glacis fall for a drop of 2.25 m in the water surface level of an irrigation channel carrying water at the rate of 60 m^3/s. The bed width and depth of flow in the channel are 30 m and 2.20 m, respectively.

Solution:

Area of flow cross-section $= 30 \times 2.20 + \frac{1}{2}(2.20)^2 = 68.42\ m^2$

Mean velocity of flow in the channel $= \frac{60}{68.42} = 0.877$ m/s

$\therefore$ Velocity head $= (0.877)^2/(2 \times 9.81) = 0.039$ m

Post-jump specific energy in the downstream (lower level) channel,

$E_2 = 2.20 + 0.039 = 2.239$ m

If the jump forms at the bed level of the downstream channel,

pre-jump specific energy = specific energy in the upstream channel + drop

$= 2.239 + 2.25 = 4.489$ m

$\therefore$ Energy loss in jump, $H_L = 4.489 - 2.239 = 2.25$ m

Using Blench's curves (Fig. 9.17) for $H_L = 2.25$ m and $E_2 = 2.239$ m,

$q = 2.85\ m^3/s/m$

Thus,

Length of crest $= \frac{60}{2.85} = 21.05$ m

This length is less than the permissible flumed width of channel which is 75% of 30 m, i.e., 22.5 m.

Therefore, provide a crest of length = 22.5 m

so that $q = 60/22.5 = 2.67\ m^3/s/m$

From $Q = 1.71\, LH^{3/2}$

$H = (Q/1.71\, L)^{2/3}$

$$= \left(\frac{60}{1.71 \times 22.5}\right)^{2/3} = 1.345 \text{ m}$$

Hence, height of crest above the upstream channel bed
= 2.239 m – 1.345 m = 0.894 m
For a meter fall, the width of crest should not be less than
2.5 H i.e., 2.5 × 1.345 = 3.363 m
Therefore, provide the crest width as 3.70 m.
The level of cistern is kept 1.25 E_2 below the level of the downstream total energy line, i.e., 1.25 × 2.239 = 2.799 m. This means that the cistern is depressed below the downstream channel bed by an amount x = 2.799 – 2.239 = 0.56 m

(The value of E_2 corresponding to q = 2.67 m^3/s/m and H_L = 2.25 m is 2.16 m).

Thus, 1.25 E_2 = 2.7 m. The lowering of the cistern level below the downstream bed would, therefore, be 2.7 – 2.16 = 0.54 m which is less than 0.56 m. Hence, the cistern is depressed by 0.56 m.
Length of cistern $L_C = 2\, E_2 = 2 \times 2.239 = 4.478 \approx 4.48$ m

Length of the downstream concrete floor (inclusive of cistern length) should be sufficient to accommodate the jump within itself.
Post-jump depth = 2.20 m

The pre-jump depth for q = 2.67 m^3/s/m and specific energy = E_2 + drop = 2.16 + 2.25 = 4.41 m is obtained from Montague's curves (Fig. 9.27) as 0.30 m.
∴ Height of jump = 2.20 – 0.30 = 1.90 m Hence,
Length of downstream floor = 5 × 1.90 = 9.50 m
On providing 2.4 m length of concrete floor upstream of the crest and the slopes of the upstream face of the crest and the glacis as 1 : 1 and 2 (H) : 1 (V), the total length of the impervious floor works out as
2.4 + 0.894 + 3.7 + 2 (0.894 + 2.25 + 0.56) + 9.50 = 23.902 m
Let the depth of the downstream cutoff be 1.5 m

$$\therefore \quad \alpha = \frac{b}{d} = \frac{23.902}{1.5} = 15.935$$

$$\lambda = \frac{1 + \sqrt{1+\alpha^2}}{2} = \frac{1 + \sqrt{1 + (15.935)^2}}{2} = 8.483$$

The exit gradient is calculated using the equation

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

The value of H for the condition of no flow with water on the upstream up to the crestlevel is (2.25 + 0.894) m, i.e., 3.144 m. The corresponding value of H, when full supply discharge is flowing, works out to only 2.25 m. Therefore, the former condition is the most critical condition for the exit gradient. Accordingly,

$$G_E = \frac{(2.25 + 0.894)}{1.5} \times \frac{1}{3.14\sqrt{8.483}} = 0.229$$

which is less than 0.25 and may be considered satisfactory. Using Eqs. (9.51) and (9.52),

$$\phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right) = \frac{1}{\pi}\cos^{-1}\left(\frac{8.483 - 2}{8.483}\right) = 0.223$$

$$\phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda - 1}{\lambda}\right) = \frac{1}{\pi}\cos^{-1}\left(\frac{8.483 - 1}{8.483}\right) = 0.156$$

Assuming a floor thickness of 0.5 m at the downstream end,

Correction for thickness (for ϕ_E) $= \dfrac{0.223 - 0.156}{1.5} \times 0.5 = 0.022$ (negative)

$\therefore$ Corrected value of $\phi_E = 0.223 - 0.022 = 0.201$

For no flow condition, the uplift pressure at the downstream end is equal to 0.201 × (2.25 + 0.894) = 0.632 m (w.r.t. the downstream bed). Ignoring the effect of the upstream cutoff (conservative approach), one can now draw the subsoil hydraulic gradient line by joining the uplift ordinate (0.632 m w.r.t. the downstream bed) at the downstream end of the impervious floor with the uplift ordinate (3.144 m w.r.t. the downstream bed, i.e., 0.894 m above the upstream end of the concrete floor) at the upstream end of the impervious floor by a straight line.

Similarly, for full supply flow condition the uplift pressure at the downstream end would be equal to 0.201 × (2.25) = 0.45 m above the downstream full supply level. The subsoil hydraulic gradient line for this case too has been shown in Fig. 10.14. Calculations for jump profile are given below:

Pre-jump Froude number, $F_1 = \dfrac{2.67/0.32}{\sqrt{9.81 \times 0.32}} = 4.7$

Therefore, from Eq. (9.20), $\bar{X} = (5.08 \times 4.7 - 7.82) \times 0.32 = 5.14$ m

Values of distance from the trough of the jump, x, and the corresponding height of the water surface profile above the trough of the jump have been computed using Fig. 9.3.

x(m)	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0
$x/\bar{X}$	0.195	0.39	0.58	0.78	0.97	1.17	1.36	1.56	1.75	1.95
y(m)	0.395	0.677	0.917	1.13	1.34	1.48	1.62	1.75	1.80	1.83

The jump profile has been plotted in Fig. 10.14.

The thickness of floor at different locations can now be computed using Eq. (9.41) on the basis of the larger of the two uplift pressures and assuming a value of G = 2.30.

Radius of curvature of upstream wing walls = 5 × 25 = 11.25 m

Provide brick pitching for a length of about 5 m upstream of the concrete floor. Providing a splay of 1 in 3 for the straight downstream expansion, the length of the downstream expansion will be 11.25 m. Provide brick pitching from the downstream end of the concrete floor to the downstream end of the expansion.

The longitudinal section of the glacis fall based on these computations is shown in Fig. 10.14.

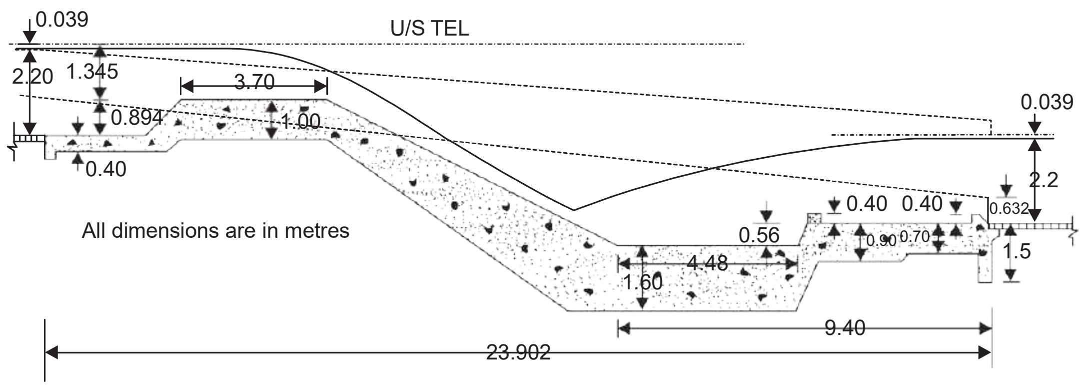


Fig. 10.14 Longitudinal section of glacis fall (Example 10.2) (not to scale)

DISTRIBUTARY HEAD REGULATOR

The distributary head regulator is constructed at the upstream end (i.e., the head) of a channel where it takes off from the main canal or a branch canal or a major distributary. The distributary head regulator should be distinguished from the canal head regulator which is provided at the canal headworks where a canal takes its supplies from a river source. The distributary head regulator serves to (i) divert and regulate the supplies into the distributary from the parent channel, (ii) control silt entering the distributary from the parent channel, and (iii) measure the discharge entering the distributary.

For the purpose of regulating the supplies entering the offtaking channel from the parent channel, abutments on either side of the regulator crest are provided. Piers are placed along the regulator crest at regular intervals. These abutments and piers have grooves (at the crest section) for the purpose of placing planks or gates. The supplies into the offtaking channel are controlled by means of these planks or gates. The planks are used for small channels in which case manual handling is possible. The span of hand-operated gates is also limited to 6 to 8 m. Mechanically-operated gates can, however, be as wide as 20 m.

An offtaking channel tends to draw excessive quantity of sediment due to the combined effects of the following:

Because of their smaller velocities, lower layers of water are more easily diverted into the offtaking channels in comparison to the upper layers of water.

Sediment concentration is generally much higher near the bed.

Sediment concentration near the banks is usually higher because of the tendency of the bottom water to move towards the banks due to difference in central and near- bank velocities of flow.

As such, if suitable steps are not taken to check the entry of excessive sediment into the offtaking channel, the offtaking channel will soon be silted up and would require repeated sediment removal.

Sediment entry into the offtaking channel can be controlled by causing the sediment to concentrate in the lower layers of water (i.e., near the bed of the parent channel upstream of the offtaking point) and then letting only the upper layers of water enter the offtaking channel. Concentration of sediment in lower layers can be increased by providing smooth bed in the parent channel upstream of the offtaking point. The smooth channel bed reduces turbulence which keeps sediment particles in suspension. In addition, steps which accelerate the flow velocity near the banks would also be useful. It should also be noted that the alignment of the offtaking channel also affects the sediment withdrawal by the offtaking channel. Hence, the alignment of the offtaking distributary channel with respect to the parent channel needs careful consideration. The angle of offtake may be kept between 60° and 80° to prevent excessive sediment withdrawal by the offtaking channel. For all important works, the alignment of offtaking channels should be fixed on the basis of model studies.

For the purpose of regulating the discharge in the distributary, it is essential to measure the discharge for which one can use gauge-discharge relationship of the distributary. However, this relationship is likely to change with the change in the channel regime. Hence, it is advantageous to use head regulator as a metering structure too.

CROSS REGULATOR

A cross regulator is a structure constructed across a canal to regulate the water level in the canal upstream of itself and the discharge passing downstream of it for one or more of the following purposes (4):

To feed offtaking canals located upstream of the cross regulator.

To help water escape from canals in conjunction with escapes.

To control water surface slopes in conjunction with falls for bringing the canal to regime slope and section.

To control discharge at an outfall of a canal into another canal or lake.

A cross regulator is generally provided downstream of an offtaking channel so that the water level upstream of the regulator can be raised, whenever necessary, to enable the offtaking channel draw its required supply even if the main channel is carrying low supply. The need of a cross regulator is essential for all irrigation systems which supply water to distributaries and field channels by rotation and, therefore, require to provide full supplies to the distributaries even if the parent channel is carrying low supplies.

Cross regulators may be combined with bridges and falls for economic and other special considerations.

DESIGN CRITERIA FOR DISTRIBUTARY HEAD REGULATOR ANDCROSS REGULATOR

The effective waterway of a head regulator should not be less than 60 per cent of the width of the offtaking canal. It should be fixed such that the mean velocity of flow at full supply condition does not exceed 2.5 m/s. The overall waterway should at least be 70 per cent of the normal channel width (at mid-depth) of the offtaking channel at the downstream of the head regulator. For cross regulators, waterway should be decided so that the resulting afflux does not exceed

0.15 m.

The crest level of a head regulator should be such that the full supply discharge of the offtaking channel can be passed even when the parent channel is running with low supplies of the order of two-thirds of the fully supply discharge of the parent channel. It should be possible to maintain full supply level in the parent channel downstream of the offtake by means of a cross regulator. The water level at the location of offtake should be computed using back water computations. The level of the crest of the head regulator is obtained by subtracting the amount of head required from the computed water level at the offtake. In any case, the crest level should not be lower than the bed level of the offtaking channel. Usually, the crest level of the head regulator is 0.3 to 0.6 m higher than the crest level of the cross regulator. The amount of head over the crest of the head regulator, H is computed from the equation (3),

$Q = CB_eH^{3/2}$ (10.23)

where, Q is the full supply discharge of the offtaking channel, and B_e is the effective width of waterway and is given as

$B_e = B_t - 2\ (NK_p + K_a)\ H$ (10.24)

Here, B_t is the overall width of the waterway (i.e., the length of the crest), N the number of piers, and K_p and K_a are contraction coefficients for piers and abutments, respectively. Values of K_p range from 0.005 to 0.02 while those of K_a range from 0.1 to 0.2.

C is a suitable discharge coefficient whose value can be taken as 1.84 for sharp-crested weirs (the crest width being less than 2/3 H) and 1.705 for broad-crested weirs (the crest width being greater than 2.5 H) for free flow conditions (3). If the flow is submerged, the values of C will have to be suitably modified depending upon the submergence ratio.

The crest of the cross regulator should be at least 0.15 m above the bed of the canal but should not be higher than 0.4 times the normal depth of the upstream canal (4). The crest width should be greater than 2/3 H and should be sufficient to accommodate the gate cill. The upstream and downstream slopes of the crest can be at slopes of 2 (H) : 1 (V). Impervious floor and cutoffs will be designed from considerations of hydraulic jump, uplift pressures, safe exit gradient, and scour depth. Similarly 'flexible' protection works at the upstream and downstreamends of the impervious floor will be provided in the form of block protection, inverted filters, and launching apron.

CONTROL OF SEDIMENT ENTRY INTO AN OFFTAKING CHANNEL When water is withdrawn by an offtaking channel from the parent canal carrying sediment- laden water, it is essential that the offtaking channel also withdraw sediment in proportion to its water discharge. For achieving proportionate distribution of sediment between the offtaking channel and the parent canal, measures such as silt vanes, groyne walls, and skimming platform are constructed.

Silt Vanes

Silt vanes (also known as King's vanes) are thin, vertical, and curved walls made of plain or reinforced concrete. The recommended dimensions of silt vanes are shown in Table 10.2 and Fig. 10.15. The dimensions are intended only as rough guides and can be used for skew offtakes also. The height of the vanes may be about one-fourth to one-third of the depth of flow in the parent canal (5). The thickness of the vanes should be as small as possible. Faces of vanes should be smooth. The spacing between the vanes may be kept about 1.5 times the vane height.

Table 10.2 Dimensions (in metres) of silt vanes (5)

W	0.60	1.2	1.8	2.4	3.0	3.6	4.6	6.0	7.6	9.0	10.6	12.0
X	1.2	1.5	2.1	2.4	3.0	3.6	4.6	5.4	6.0	7.0	7.8	8.5
Y	0.6	1.2	1.5	1.8	2.4	2.7	3.0	4.0	5.2	6.0	6.6	7.6
Z	1.2	1.2	1.5	1.8	2.4	2.7	3.0	3.6	4.2	5.2	5.8	6.6
R	9.0	9.0	10.0	12.0	18.0	21.0	24.0	30.0	35.0	44.0	50.0	57.0

Note: See Fig. 10.15 for meaning of symbols used in this table.

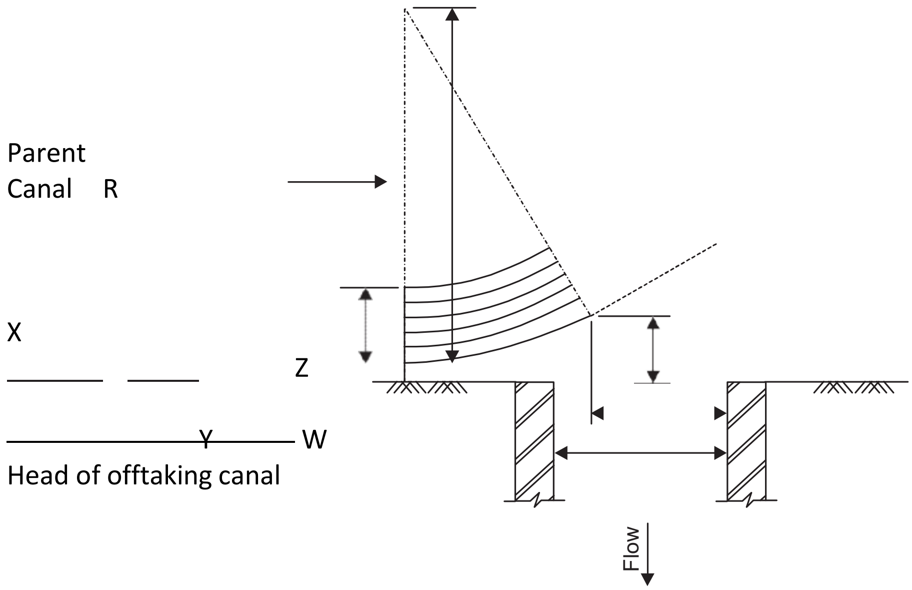


Fig. 10.15 Silt vanes (5)

Reverse vanes (Fig. 10.16) may have to be provided in cases in which the width of the parent canal is small and the sediment deflected towards the edge of the parent canal is likely to be deposited there due to low velocities.

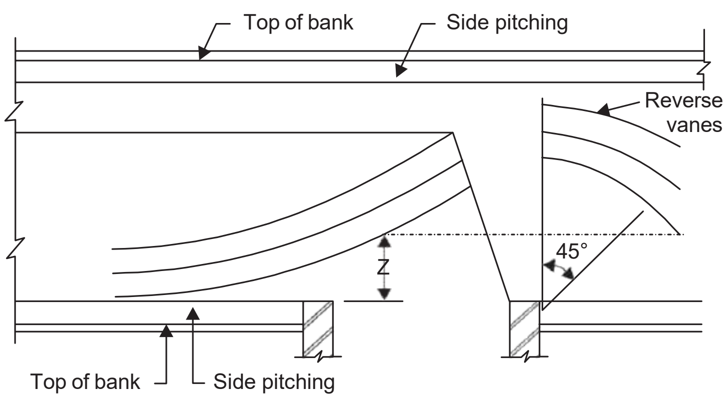


Fig. 10.16 Reverse vanes (5)

Groyne Wall (Curved Wing)

A curved vertical wall (also known as Gibb's groyne wall) extending from the downstream abutment of the offtaking channel into the parent channel (Fig. 10.17) causes the offtaking channel to draw its share of sediment load. The wall should extend at least up to 3/4 of the width of the offtake. It may, however, preferably extend up to the upstream abutment of the offtaking canal. The nose of the wall should be pointed and vertical, and thickness of the wall should increase gradually. The top of the wall is kept at least 30 cm above the full supply level of the parent channel (6). If the offtaking channel has a slope milder than that of the parent channel, the curved wall may not be useful to the desired extent, and has to be employed in conjunction with sediment vanes (6).

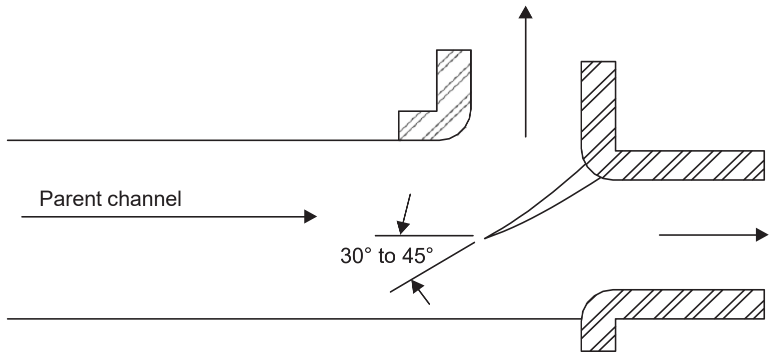


Fig. 10.17 Curved wing wall (6)

Skimming Platform

A skimming platform consists of an RC slab placed horizontally in the parent channel in front of the offtake (Fig. 10.18). It works on the principle of sediment excluder (Sec. 13.11) and is suitable only where the parent channel is deep (say 2 m or more) and the offtake is comparatively small (7).

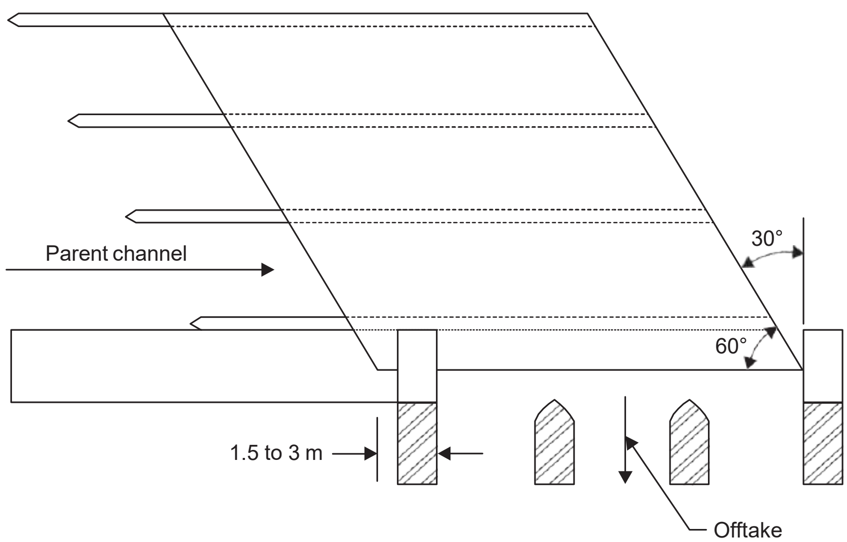


Fig. 10.18 Skimming platform (7)

CANAL ESCAPES

A canal escape is a structure to dispose of surplus or excess water from a canal. A canal escape essentially serves as a safety valve for the canal system. It provides protection of the canal against possible damage due to excess supplies which may be on account of either a mistake in releasing water at headworks, or a heavy rainfall due to which there may be sudden reduction in demand, making the cultivators close their outlets. The excess supply makes the canal banks vulnerable to breaches or dangerous leaks and, hence, provision for disposing of excess supply in the form of canal escapes at suitable intervals along the canal is desirable. Besides, emptying the canal for repairs and maintenance and removing a part of sediment deposited in the canal can also be accomplished with the help of the canal escapes. The escapes are usually of the following types (8):

Weir or surface escape: These are weirs or flush escapes constructed either in ma- sonry or concrete with or without crest shutters which are capable of disposing of surplus water from the canal.

Sluice escapes: Sluices are also used as surplus escapes. These sluices can empty the canal quickly for repair and maintenance and, in some cases, act as scouring sluices to facilitate removal of sediment.

Location of escape depends on the availability of suitable drains, depressions or rivers with their bed level at or below the canal bed level for disposing of surplus water through the escapes, directly or through an escape channel.

CROSS-DRAINAGE STRUCTURES

NEED OF CROSS-DRAINAGE STRUCTURES

Aligning a canal on the watershed of an area is necessary so that water from the canal can flowby gravity to fields on both sides of the canal. However, a canal taking off from a river at A (Fig. 11.1) has to necessarily cross some streams or drainages (such as at a, b, c, and d in the figure) before it can mount the watershed of the area at B. In order to carry a canal across the streams, major cross-drainage structures have to be constructed. Once the canal is on the watershed at B, usually no cross-drainage structure is required except in situations when the canal has to leave a looping watershed (such as DEF in Fig. 11.1) for a short distance between D and F, and may cross tributaries (as at e and f). Cross-drainage structures are constructed to negotiate an aligned channel over, below, or at the same level of a stream (1).

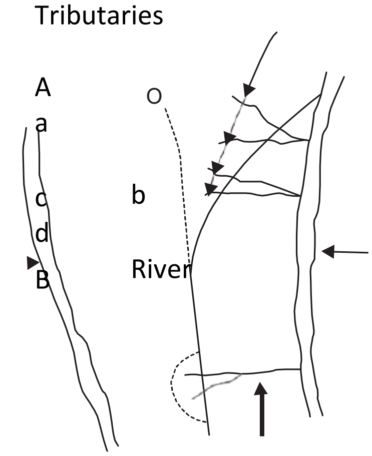


Fig. 11.1 Canal alignment between offtake and watershed

TYPES OF CROSS-DRAINAGE STRUCTURE

The cross-drainage structures can be classified under three broad categories depending on whether the structure is built to negotiate a carrier channel over, below, or at the same level as the stream channel.

Structures for a Carrier Channel Over a Natural Stream

The structures falling under this category are aqueducts and siphon aqueducts. Maintenance of such structures is relatively easy as these are above ground and can be easily inspected. When the full supply level (FSL) of a canal is much higher than the high flood level (HFL) of a stream which, in turn, is lower than the bottom of the canal trough, the canal is carried over the stream by means of a bridge-like structure, which is called an aqueduct. The stream water passes through the space below the canal such that the HFL is lower than the underside of the canal trough, Fig. 11.2

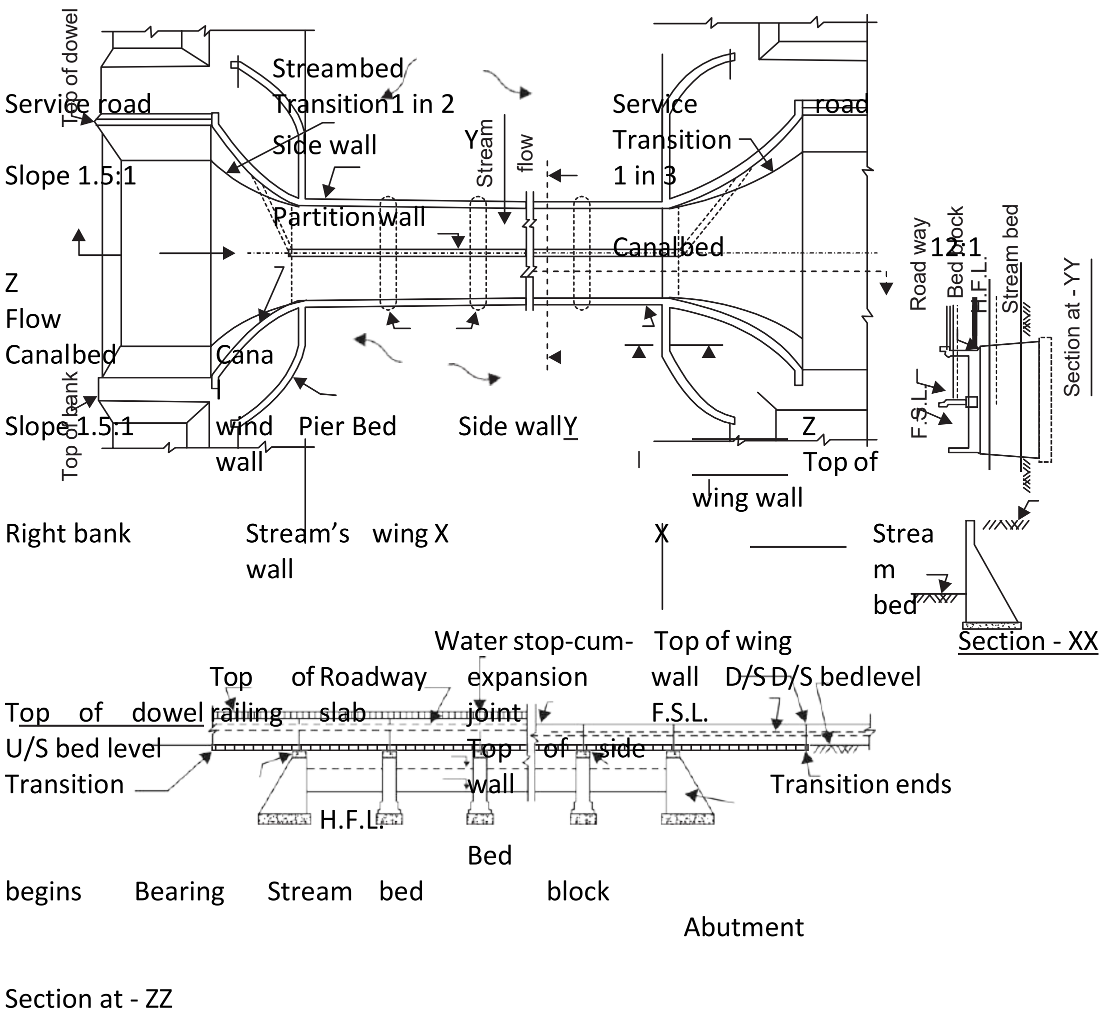


Fig. 11.2 Typical plan and section of an aqueduct (1)

Siphon aqueduct (Fig. 11.3) is an aqueduct in which the bed of the stream is depressed when it passes under the canal trough, and the stream water flows under pressure below the canal. In such aqueducts, the stream bed is usually provided with a concrete or masonry floor.

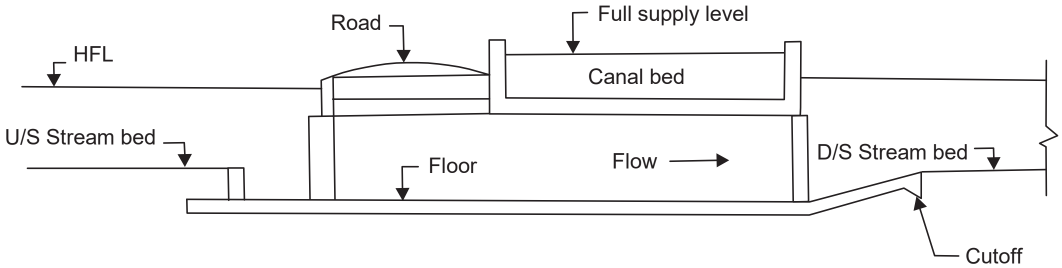


Fig. 11.3 Siphon aqueduct

Aqueducts and siphon aqueducts are further classified into the following types:

Type I: In this type of structure, the earthen canal banks are carried as such and, hence, the culvert length (i.e., the length of barrels through which the stream water is passed under the canal) has to be long enough to support the water section as well as the earthen banks of the canal [Fig. 11.4 (a)]. In this type of structure, the canal section is not flumed and remains unaltered. Hence, the width (across the canal) of the structure is maximum. This type of structure, obviously, saves on canal wings and banks connections and is justified only for small streams so that the length (along the canal) of the structure is small. An extreme example of such a structure would be when the stream is carried by means of a pipe laid under the bed of the canal.

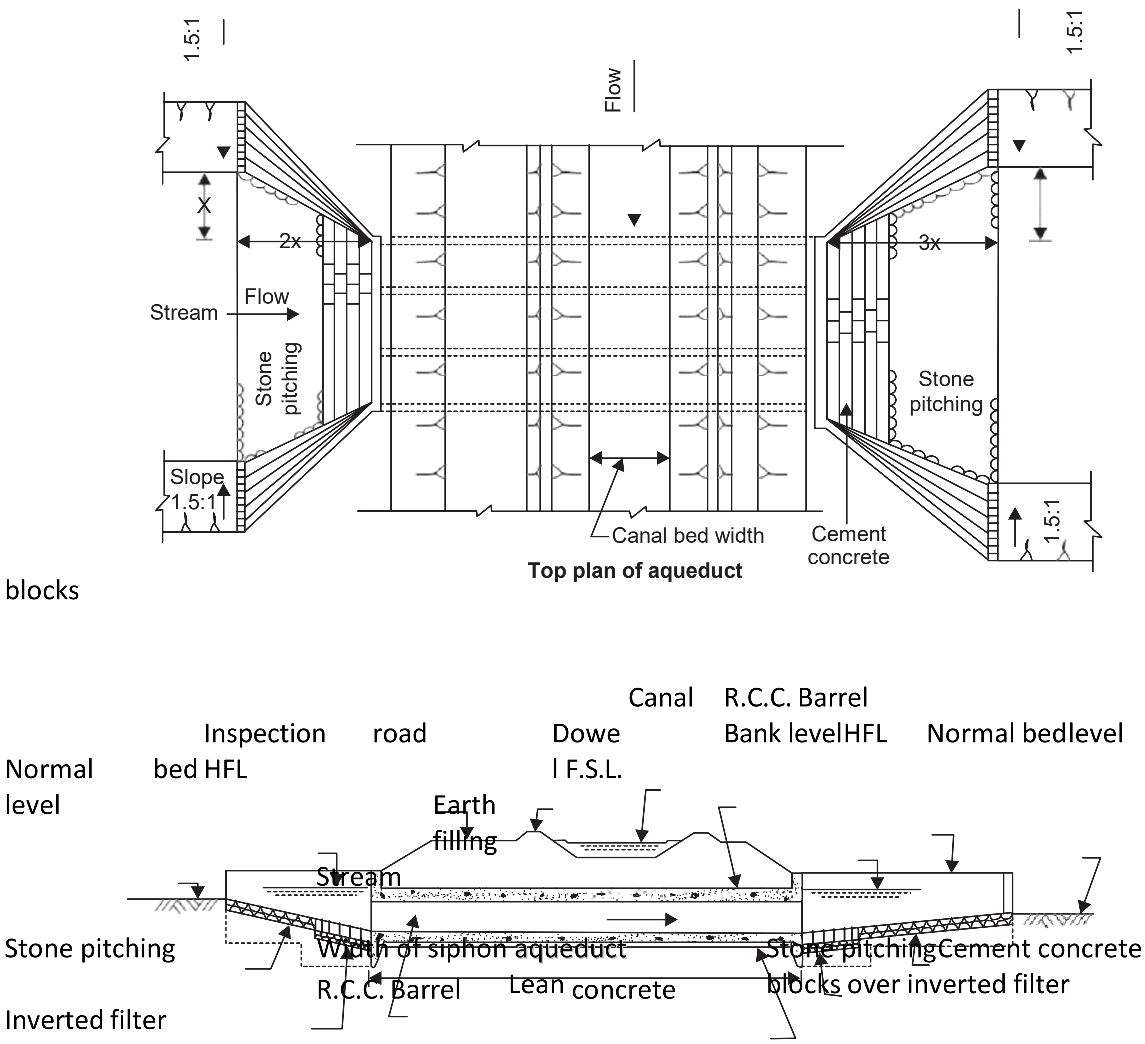


Fig. 11.4 (a) Typical plan and section of a siphon aqueduct (Type I) (1)

Type II: This type of structure is similar to the Type I with a provision of retaining walls to retain the outer slopes of the earthen canal banks [Fig. 11.4 (b)]. This reduces the length of the culvert. This type of construction can be considered suitable for streams of intermediate size.

Type III: In this type of structure, the earthen canal banks are discontinued through the aqueduct and the canal water is carried in a trough which may be of either masonry or concrete [Fig. 11.4 (c)]. The earthen canal banks are connected to the respective trough walls on their sides by means of wing walls. The width of the canal is also reduced over the crossing. In this type of structure, the width of the structure is minimum and, hence, the structure is suitable for large streams requiring considerable length of aqueduct between the abutments.

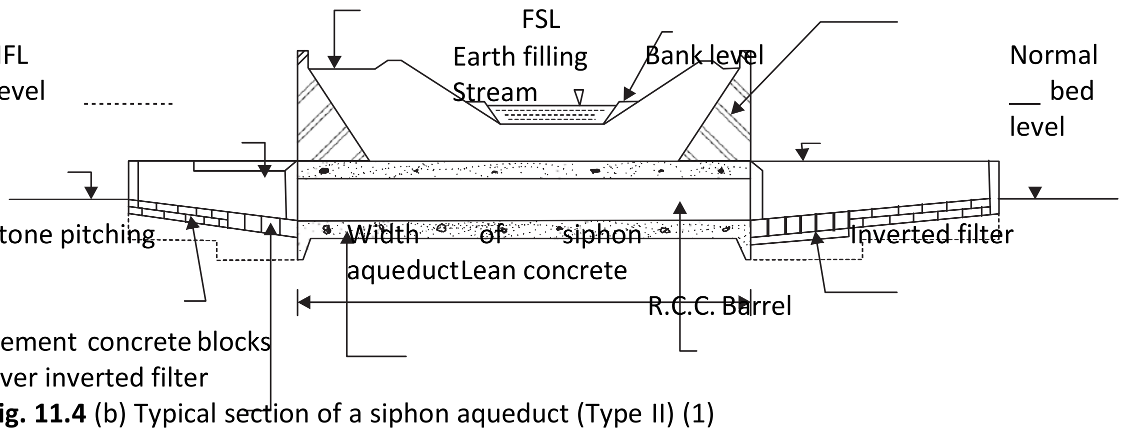


Fig. 11.4 (b) Typical section of a siphon aqueduct (Type II) (1)

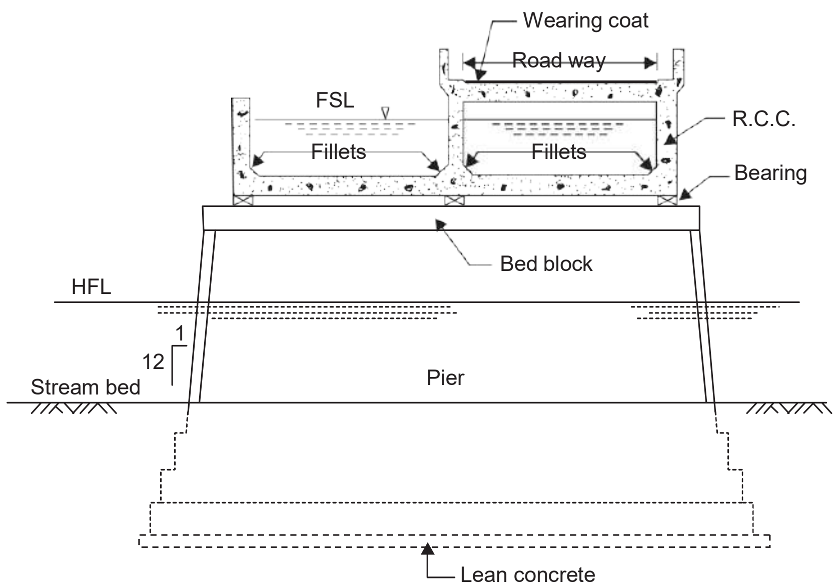


Fig. 11.4 (c) Typical section of an aqueduct (Type III) (1)

Structures for a Carrier Channel Underneath a Natural Stream

The structures falling under this category are superpassages and siphons. The maintenance ofsuch structures is relatively difficult as these are not easily accessible.

A superpassage [Fig. 11.5] is like an aqueduct, but carries the stream over the canal. The canal FSL is lower than the underside of the stream trough and, hence, the canal water flows with a free surface.

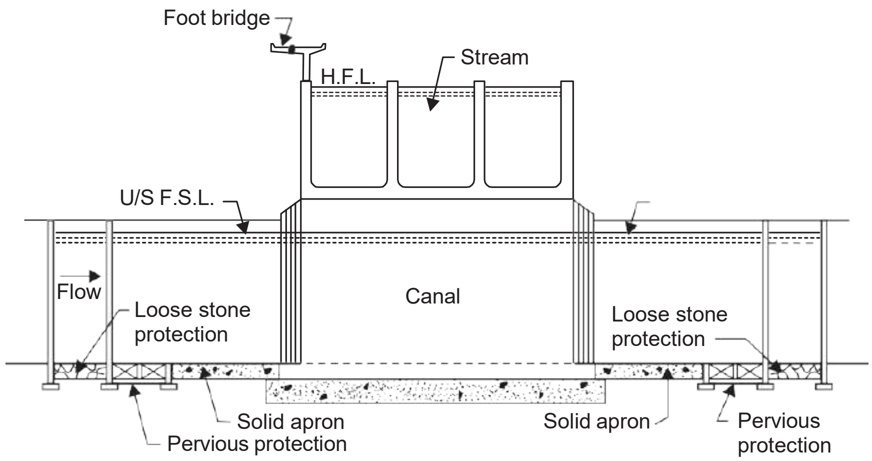
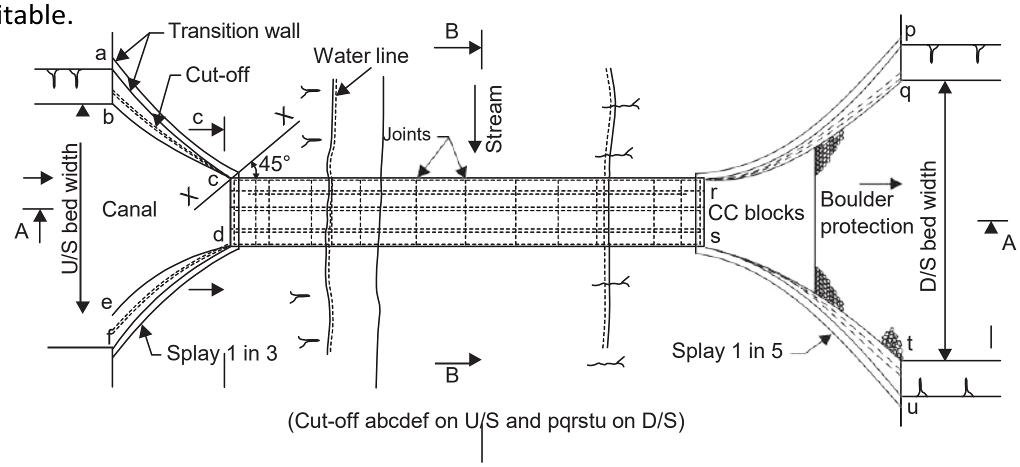
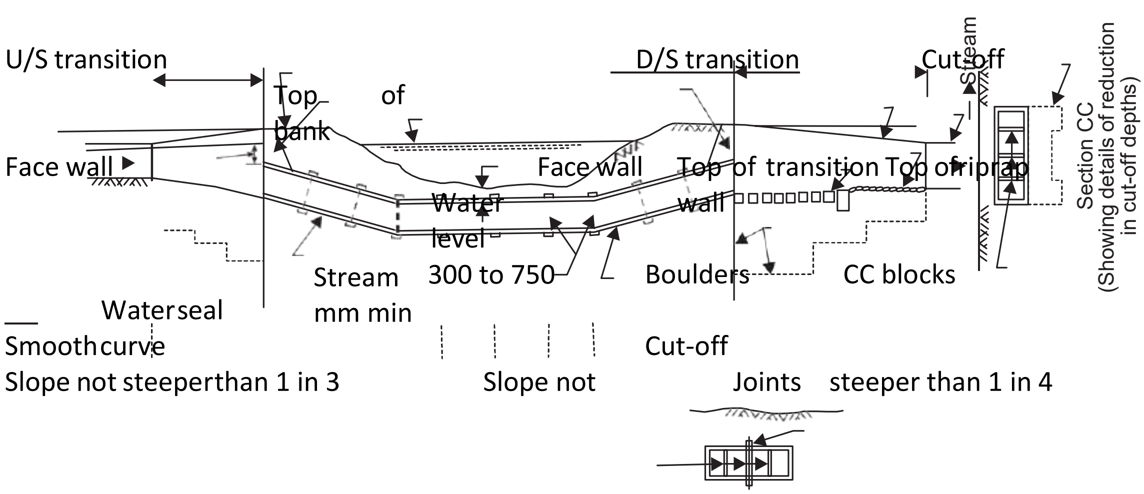


Fig. 11.5 Typical cross-section of a superpassage (1)

A siphon [Fig. 11.6] carries the canal water under pressure through barrels below the stream trough. For siphoning small discharges, precast RCC pipes will be economical. For siphoning higher discharges, horse-shoe-shaped rectangular or circular barrels, either single or multiple, are adopted. Roofs of rectangular barrels are, at times, arch-shaped for economy. For discharges under high pressures, circular or horse-shoe-shaped barrels are more suitable.



PLAN



Partitionwalls of barrel

Section AA

Partition walls of barrel

Rubber water stops at top and bottom may be provided if numbers of barrel are large

Section BB

Fig. 11.6 Profile of a typical canal siphon (1)

Structures for Carrier Channel Crossing a Natural Streamat the Same Level

Structures falling under this category are level crossings and inlets. Inlets are, at times, combined with escapes. When the canal and the stream meet each other at practically the same level, a level crossing [Fig. 11.7] is provided. Level crossings involve intermixing of the canal and stream waters. They are usually provided when a large-sized canal crosses a large stream which carries a large discharge during high floods, and when siphoning of either of the two is prohibitive on considerations of economy and non-permissibility of head loss through siphon barrels (1). A barrier with its top at the canal FSL is constructed across the stream and at the upstream end of the junction. The regulators are provided across the stream and canal at the downstream junctions of the level crossing. These regulators control the flow into the canal and stream downstream of the crossing. This type of arrangement is also useful in augmenting the canal supplies with the stream discharge.

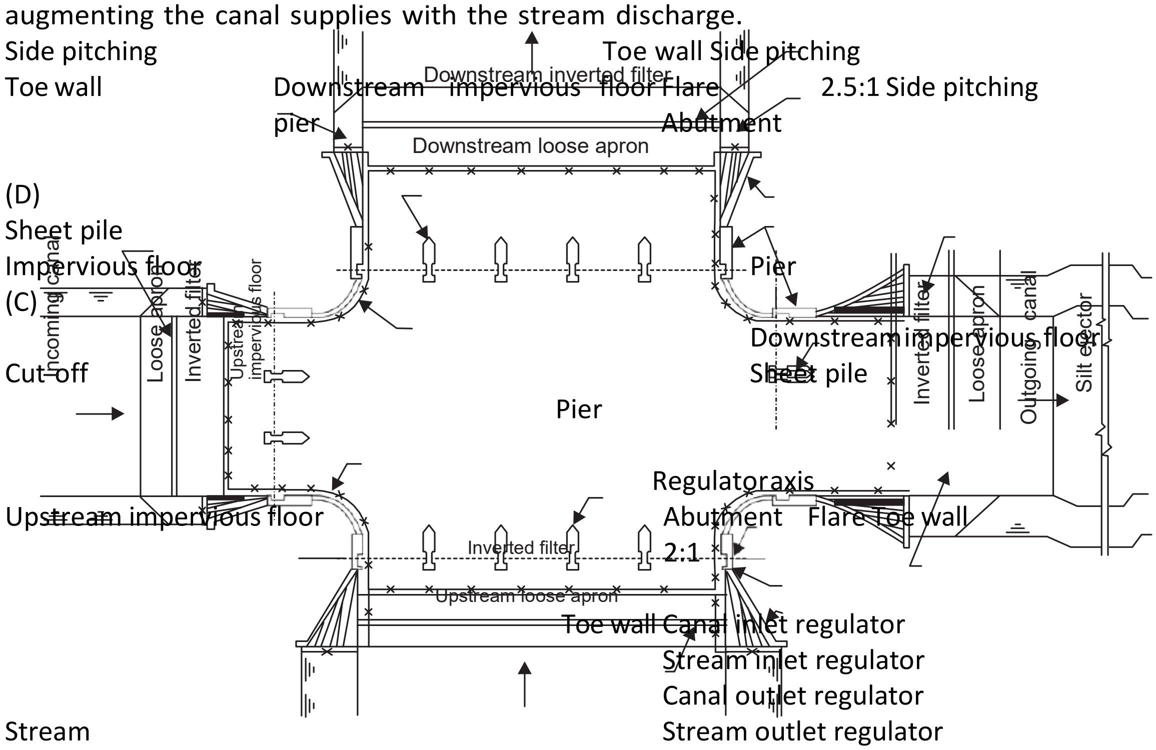


Fig. 11.7 Typical layout plan of a level crossing (1)

When the stream is dry, the stream regulator is kept closed and the canal regulator is opened so that the canal water flows in the canal itself without interruption. When the stream is bringing water, it mixes with the canal water and the stream regulator is used to dispose of that part of the stream water which is not used to augment the canal supply.

If a small and relatively sediment-free stream meets the canal at practically the same level (or its bed level is higher than the canal FSL) then an inlet is provided. An inlet is a structure consisting of an opening in a canal bank, suitably protected, to admit upland stream water into the canal. Inlets are constructed only if the stream discharge is too small and does not carry large quantity of sediment. Inlets do not have a regulator and, hence, the stream bed should be higher than the canal FSL. An inlet consists of a fall or a pitched slope confined within wing walls to guide the stream water into the canal. An inlet simply allows the stream water to be taken into the canal. If the stream water so taken into the canal is appreciable in quantity, it is allowed to flow out at a suitable site downstream (along the canal) of the inlet. The outlet is generally combined with some other structure for economic reasons, but at times only an inlet (and no outlet) is provided.

Sometimes a small stream is diverted into a larger stream and a cross-drainage structure for the combined discharge is provided at a suitable site.

SELECTION OF SUITABLE CROSS-DRAINAGE STRUCTURE

Relative differences in bed and water levels of a canal with those of the crossing stream as well as their discharges are the main factors for deciding the type of cross-drainage structure at a site. By suitably changing the alignment of the canal between offtaking point A and the watershed (Fig. 11.8) the relative difference between the bed levels of the tributaries and the canal at the crossing site can be altered. Consider three possible alignments ABC, ADE, and AFG of a canal taking off from a river at A and intersecting a tributary HBDFI at B, D, and F before mounting the watershed at C, E, and G, respectively (Fig. 11.8). The distances AB, AD, and AF are almost the same and, hence, the canal reaches the crossing site with its bed more or less at the same level. But, the bed levels of the tributary at B, D, and F are significantly different due to the slope of the tributary. Obviously, the bed level of the tributary is the highest at B, and the lowest at F in the reach BDF. Thus, if F is a suitable crossing site for aqueduct, site D may necessitate the construction of a siphon aqueduct or level crossing and site B may require the construction of a siphon or a superpassage. Thus, the type of cross- drainage structure can be changed by suitably altering the crossing site.

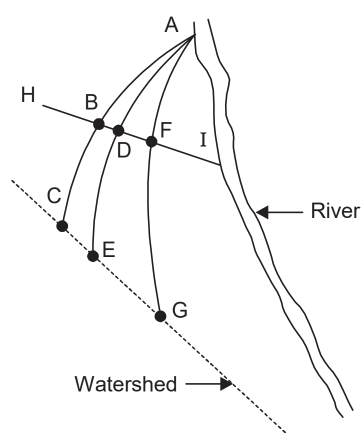


Fig. 11.8 Different canal alignments between offtake and watershed

When the crossing site is such that the canal FSL is well above the stream HFL the choice between aqueduct and siphon aqueduct is made depending on the stream discharge. For larger stream discharges (i.e., when the stream bed is much wider) an aqueduct is more suitable than a siphon aqueduct which requires lowering of the stream bed by a drop. Besides being costly, lowering of the bed may result in silting on the lowered stream bed which increases the risk of failure. However, an aqueduct necessitates heavy canal embankments towards the crossing (Fig. 11.9). This is due to the wide flood cross-section of streams in plains and the requirements that the canal must be well above the HFL, and the aqueduct has to be constructedin a smaller part of the cross-section of the stream.

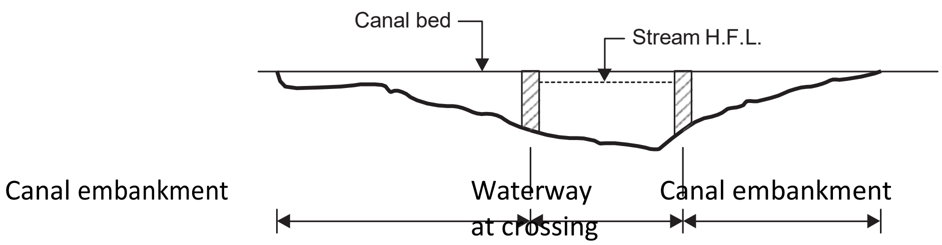


Fig. 11.9 Canal embankments near crossing site

Siphon aqueducts are more suitable when the stream size is small compared to the canal size. In case of siphon aqueducts, the relative differences of water and bed levels of the canal with those of the crossing stream is small and, hence, embankments of only small heightare required.

If the stream HFL is well above the canal FSL, superpassage is generally preferred in comparison to siphon as the latter involves considerable head loss in the canal. In addition, the construction of a siphon under a stream with an erodible bed requires heavy protection works. The foundations of the superpassage and siphon have to be carried up to much below the erodible bed of the stream. A separate bridge across the stream trough has to be provided to carry the canal road across the stream. The construction of these structures is relatively difficult and costly due to the requirements of extensive training works and large stream trough to carry the high flood discharge. If the canal serves navigation needs also, then sufficient headway should be provided for the passage of boats.

If the bed and water levels of the canal and stream at the crossing site are approximately the same, a level crossing is provided. Sometimes, due to prohibitive costs of siphons andsiphon aqueducts, the canal alignment between the offtake and watershed is suitably altered so that the level crossing can be provided at the crossing site. The initial cost of a level crossing is generally much lower than the cost of other cross-drainage structures. Also, the perennial discharge of the stream can be diverted to the canal to provide additional irrigation. However, the level crossing requires permanent staff for continuous watch, maintenance, and operation of gates. Also, when the stream is passing the high flood discharge, the canal may have to be closed down to prevent the sediment load of the stream from entering the canal and silting it. Further, if the canal FSL is higher than the general ground level, the HFL of the stream would increase on the upstream side of the crossing site causing submergence of the land. To prevent such submergence of the land, marginal banks are provided.

In addition to the above factors, the topography of the terrain, foundation conditions, regime of the stream, and dewatering requirements would also affect the choice of the type of cross-drainage structures. Detailed examination of the terrain topography and the foundation is necessary to locate a stable reach of the stream with good foundations and permitting preferably a right-angled crossing. For streams carrying high sediment discharge, the possibility of chocking up of the siphon and the effect of fluming of the stream should be kept in mind. Dewatering of foundations is necessary in the construction of foundations for cross-drainage structures. An accurate estimate of the cost and method of dewatering must be worked out when design involves laying of foundations below the ground water table.

DESIGN OF CROSS-DRAINAGE STRUCTURES

Before undertaking detailed designs of any important cross-drainage structure, collection of relevant field data is required. Besides, a note regarding the several alternative alignments surveyed and reasons for final selection of a particular crossing site is also necessary. A location map of the site along with the results of subsurface explorations of the site, and stream cross- sections at different locations around the site should also be prepared. The following specific hydraulic data regarding the canal and stream should be made available.

Canal

Discharge, depth of flow, and water level at full supply,

Bed width,

Canal bed slope and the water surface slope,

Levels of canal bed and the top of canal banks,

Channel cross-section,

Characteristics of material of the bed and sides of the channel, and

Width of roadway and type and class of IRC loading.

Stream

Extent and nature of stream and its catchment,

Detailed records of rainfall in the catchment,

Maximum observed discharge,

Maximum flood level and water surface slope as observed under the highest flood condition at the proposed site,

Site plan of the proposed crossing including contours,

Log of borehole or trial pit data.

Information about the sediment load,

Characteristics of the bed material including Manning's n and silt factors.

Longitudinal section of the stream for suitable distance upstream and downstream of the chosen site for cross-drainage structure depending upon the site conditions,

Cross-section of the stream for about 100 to 300 m upstream and downstream of the site at intervals of 10 to 50 m,

Waterways provided in road/railway bridges or other hydraulic structures of the area,

Spring water levels at the crossing site.

Any cross-drainage structure should preferably be located in a straight reach of the stream crossing the canal at right angle as far as possible. The alignment of the canal should also be such that it results in minimum lengths of embankments (for aqueduct and siphon aqueduct structures). If required, the site of the structure may even be shifted away from the existing stream channel, when it is possible to divert the channel and also keep it there by reasonable training works. One obvious advantage of such an alternative would be that the construction will be carried out in dry conditions.

The design discharge from any cross-drainage structure would depend upon the size and importance of the structure. The failure of a large cross-drainage structure may result in the submergence of considerable cultivable and residential areas besides interrupting irrigation and resulting in reduction of crop yield over a large area. Therefore, the design discharge for very large cross-drainage structures should be based on the maximum probable flood from the maximum probable storm. However, for small cross-drainage structures, it would be very uneconomical to use the highest peak flood for design. The failure of a small cross-drainage structure would not cause much submersion, and interrupt irrigation only to a marginal extent. In the long run, it may prove to be more economical to repair or even replace relatively small cross-drainage structures at long intervals than to spend very large sums in order to provide for the highest peak flood.

For major cross-drainage structures, the design flood discharge can be taken as the discharge of a 1-in-50 to 1-in-100-years flood. For small cross-drainage structures, however, the design flood discharge may correspond to a 10- to 25-years frequency flood with increased afflux. In cases of important structures, an additional margin of safety is provided in the foundation design and fixation of the freeboard to take care of the unexpected and unforeseen nature of flood intensities (Table 11.1).

Table 11.1 Suggested increase in design discharge (1)

Catchment area (Km^2)	Increase in discharge (%)
More than 25,000	0—10
5000-25,000	10—20
500-5000	20—25
Less than 500	25—30

The type of foundation for cross-drainage structures will depend primarily on the depth of scour calculated from Lacey's equation [Eq. (8.32) or Eq. (8.33)], and the bearing capacity of the soil. The depth of scour around piers is taken as twice the depth of scour calculated from Lacey's equation. In alluvial streams, a well foundation is usually provided where deep foundation is required. With the provision of an impervious floor (necessary for siphon and siphon aqueduct) along with cutoff walls, the depth of foundation may be reduced. The floor itself may be designed as either a gravity floor or a raft. The floor is designed to resist the total uplift pressure caused by subsoil water and the water seeping from the canal. The uplift pressure is counterbalanced by the dead weight of the gravity floor. The worst condition occurs when there is no water in the barrel and, hence, the weight of water in the barrel is not included in the design. At times, it may be economical to design the floor as a raft so that the uplift is counterbalanced by the entire weight of the superstructure. The spacing of the piers (i.e., the span) depends on structural and economic considerations. Fewer piers (i.e., longer span) are preferable at sites which require costly foundation.

In case of siphon aqueducts and siphons, the drop at the upstream end of the culvert may be vertical (generally economical) or sloping. But, at the downstream end of the culvert, the rise should always be at a slope flatter than 1 in 4 so that the bed load can be moved out of the siphon barrel. The culvert floor should extend upstream of the barrel inlet by a distance equal to the difference between the HFL and the culvert floor level. Barrel inlet should be bell-mouthed to reduce the head losses.

A suitable arrangement has to be provided to pass the service road across the stream. This requirement does not pose much of a problem in cross-drainage structures of type I and II in which earthen embankments are continued. In cross-drainage structures of type III, the simplest arrangement is to carry the road on either side (or only on one side for economic reasons) by providing slabs and arches on either side (or on one side) of the canal trough.

The sides of the canal trough are generally designed as beams in reinforced concrete structures. The bottom slab is suspended from these beams. Additional beams, if required, are projected into the canal to divide the canal trough into a number of parallel channels. For wider troughs having intermediate beams, the service road may be provided on one of the compartments. Canal troughs of the smaller width can be designed as a hollow box girder and the service road can be provided on the top slab.

The forces acting on a cross-drainage structure consist mainly of the hydrostatic and uplift pressures, earth pressures, and dead weights and live loads of construction equipment and traffic. The overall design of the structure should be such that the total weight of the structure is as small as possible. Possibilities of differential settlement and excessive scour during floods must always be kept in mind while designing the foundations.

Wing walls of the stream are suitably connected to high ground. The stream should be guided towards the structure by means of suitable river training works. Similarly, the canal banks, adjacent to the crossing, should be protected by measures such as pitching, launchingapron, etc., wherever necessary.

The design of any cross-drainage structure, like any other hydraulic structure, includes hydraulic, structural and foundation aspects. The hydraulic aspects include the surface and subsurface flow considerations. The surface flow determines the configuration of the structure so that the structure is economic and functionally efficient. The surface and the subsurface flow considerations enable determination of the following:

Waterway and headway of the stream,

Head loss through the cross-drainage structure,

Fluming or contraction of the canal waterway,

Uplift pressures on the trough,

The uplift pressures on the culvert floor,

The exit gradient, and

Protection works.

The hydraulic aspects of the design of cross-drainage structures have been dealt with in the following paragraphs.

WATERWAY AND HEADWAY OF THE STREAM

Economic and safety considerations limit the waterway of the stream. Lacey's regime perimeter equation [Eq. (8.29)] can be used for determining the permissible waterway for structure without rigid floor. In structures with rigid floors, however, the waterway can be further reduced, but keeping the flow velocities within permissible limits (Table 11.2).

The purpose of providing waterway equal to the Lacey's perimeter is to let a stable channel develop between the training banks (i.e., the guide banks) upstream of the cross-drainage structure. At the site of the structure, the piers for the structure will reduce the available clear waterway. For small structures, the permissible reduction in clear waterway is up to 20%. It should be noted that at the site of the structure, the regime conditions do not exist and the reduction in clear waterway up to 20% is not a cause of concern if the scour is calculated using the increased discharge per unit length of clear waterway.

Table 11.2 Maximum permissible velocities (1)

Type of floor	Maximum permissible velocity (m/s)
Steel-and cast iron-lined face	10
Concrete-lined face	6
Stone masonry with cement pointing	4
Stone (or brick) masonry with cement plastering	4
Hard rock	4
Brick masonry with cement pointing	2.5

In aqueducts, the height of barrels is fixed such that the canal trough is about 0.6 m above the HFL of the stream. The requirement of the distance between the full supply level of canal and the bottom of the stream trough is, however, less in case of superpassage as FSL is relatively a certain quantity. But, in siphon aqueducts, the required waterway area is calculated on the basis of permissible scouring velocity (generally 2-3 m/s) through the barrels. Velocities higher than the permissible velocity will result in higher afflux upstream of the structure resulting in higher and longer marginal banks.

In aqueducts as well as siphon aqueducts, it is necessary to have sufficient headway between the bed level of the stream (downstream of the crossing) and underside of the culvert roof. This headway should be at least 1 m or half the height of the barrel, whichever is less. In the absence of a clear headway, there will exist the risk of the barrels being blocked because of silting. To fulfil the requirements of the headway, the stream bed may have to be lowered by providing a fall upstream of the crossing.

HEAD LOSS THROUGH CROSS-DRAINAGE STRUCTURES

The HFL of a stream downstream of a cross-drainage structure remains unchanged but the upstream water level will rise by an amount equal to the head loss (or afflux) due to the flow in the barrels of the cross-drainage structure. The length and top elevation of the guide banks upstream of the structure will depend on the raised HFL. Depending upon the flow conditions in the barrels, the broad-crested weir discharge formula or orifice discharge formula can be used for calculating afflux (1).

Alternatively, one can determine the afflux Δh from the following empirical formula proposed by Yarnell (2):

$$\frac{\Delta h}{h_3} = KF_3^2[K + 5F_3^2 - 0.6](\alpha + 15\alpha^4) \quad (11.1)$$

Here, h_3 is the depth of flow sufficiently downstream of the piers, and F_3 is the corresponding Froude number. The term α is the ratio of the width of pier with the spacing (centre-to-centre) of the piers, and K is dependent on the shape of the pier as given in Table 11.3.

Table 11.3 Values of K in Eq. (11.1)

Shape of pier	K
Semicircular nose and tail	0.90
Nose and tail formed of two circular curves each of radius equal to twice the pier width and each tangential to pier face	0.90
Twin-cylinder piers with connecting diaphragm	0.95
Twin-cylinder piers without diaphragm	1.05
90°-triangular nose and tail	1.05
Square nose and tail	1.25

The total head loss, h_L, for a flow through siphon or siphon aqueduct can be obtained as the sum of the losses at the inlet and outlet and the friction loss. If approach velocity V_a is also significant, the approach velocity head may also be taken into account. Assuming the downstream velocity head to be negligible, the head loss (or afflux) h_L is expressed as,

$$h_L = \left[1 + f_1 + f_2 \frac{L}{R}\right]\frac{V^2}{2g} - \frac{V_a^2}{2g} \quad ...(11.2)$$

Here, L is the length of the barrel, R the hydraulic radius of the barrel section, V the velocity of flow through the barrel section, and V_a is the approach velocity which is generally
neglected. f_1 is the entry loss coefficient whose value is 0.505 for an unshaped entrance and
0.08 for bell-mouthed entrance (1). f_2 is a coefficient similar to the friction factor and is equal to a [1 + (b/R)]. Here, R is expressed in metres. The values of a and b for different types of barrel surface are given in Table 11.4.

Table 11.4 Values of a and b for f_2 (3)

Barrel surface	a	b
Smooth iron pipe	0.00497	0.025
Incrusted iron pipe	0.00996	0.025
Smooth cement plaster	0.00316	0.030
Ashlar or brickwork or planks	0.00401	0.070
Rubble masonry or stone pitching	0.00507	0.250

In order to minimise the head loss and afflux, the barrel surface should be made smooth and the entrance of the barrel bell-mouthed.

DESIGN OF TRANSITIONS FOR CANAL WATERWAY

The cost of an aqueduct or siphon aqueduct will depend on its width and other factors. The width of the aqueduct is, therefore, reduced by contracting the canal waterway. However, the canal waterway is not contracted in case of earthen banks and, hence, contraction of the canal waterway is considered only in structures of type III [Fig. 11.4 (c)]. There is, however, a limit up to which a canal can be contracted or flumed. The fluming should be such that it should not result in supercritical velocity in the canal trough. The supercritical velocity in the canal trough may cause the formation of a hydraulic jump before the supercritical flow of the canal trough meets the subcritical flow of the normal canal section. The jump formation would result in additional loss of energy and large forces on the structure. Also, the lengths of transitions increase with the amount of reduction in the canal width. The fluming should be such that the cost of additional length of transition is less than the savings in cost on account of the reduction in the width of the aqueduct (or siphon aqueduct). The canal should not be flumed to less than 75 per cent of bed width. If the velocity and the head loss permit, greater fluming may be allowed ensuring subcritical flow conditions in the flumed canal (1). Proper transitions need to be provided between the flumed portion and normal section of the canal. A minimum splay of 2 : 1 to 3 : 1 for the upstream contracting transition, and 3 : 1 to 5 : 1 for the downstream expanding transition is always provided (1).

An increase in the depth of flow in the transitions and also in the canal trough will result in deeper foundation and higher pressures on the roof as well as higher uplift on floor of the culvert. The canal trough and transitions are, therefore, generally designed keeping the depth of flow the same as in the normal section.

While constructing a cross-drainage structure, it is often required to connect two channels of different cross-sectional shapes. Usually, a trapezoidal channel is required to be connected to a rectangular channel or vice versa. A channel structure providing the cross-sectional change between two channels of different cross-sectional shapes is called channel transition or, simply, transition. Transition structure should be designed such that it minimises the energy loss, eliminates cross-waves, standing waves and turbulence, and provides safety for the transition as well as the waterway. Besides, it should be convenient to design and construct the structure. These transitions are usually gradual for large and important structures so that the transition losses are small. Abrupt transitions may, however, be provided on smaller structures.

For a contracting transition (Fig. 11.10), the flow is accelerating and as such any gradual contraction which is smooth and continuous should be satisfactory. A quadrant of an ellipse with its centre on bb can be chosen for determining the profile of the bed line. For a splay of 1in m_1, the bed line profile can be written as

$$\left[\frac{x}{0.5m_1(B_c - B_f)}\right]^2 + \left[\frac{y}{0.5(B_c - B_f)}\right]^2 = 1 \qquad (11.3)$$

Side slopes and the bed elevation may be linearly varied. The value of m_1 should be kept higher than 3. In the case of an expanding transition, however, more care must be exercised. Following discussions pertain to the design of expanding transitions.

Irrigation channels carry subcritical flow and the hydraulic design of gradual transitions for subcritical flow requires: (i) prediction of flow conditions at section f-f (Fig. 11.10) for the given size and bed elevation of the flume and also the flow condition at the exit section c-c, and

(ii) determination of the boundary shape and flow conditions within the transition. For given flow conditions in the exit channel, the depth and velocity of flow (and, hence, the energy loss) within the transition are governed by the following three boundary variations:

$B = f_1(x)$ (11.4)

$\Delta z = f_2(x)$ (11.5)

and $m = f_3(x)$ (11.6) where, B = the bed width of the transition at distance x from the flume end of the transition

(Fig. 11.10),

Δz = change in bed level (with respect to the flume bed) within the transition, and m = channel side slope (i.e., m(H) : 1 (V) within the transition).

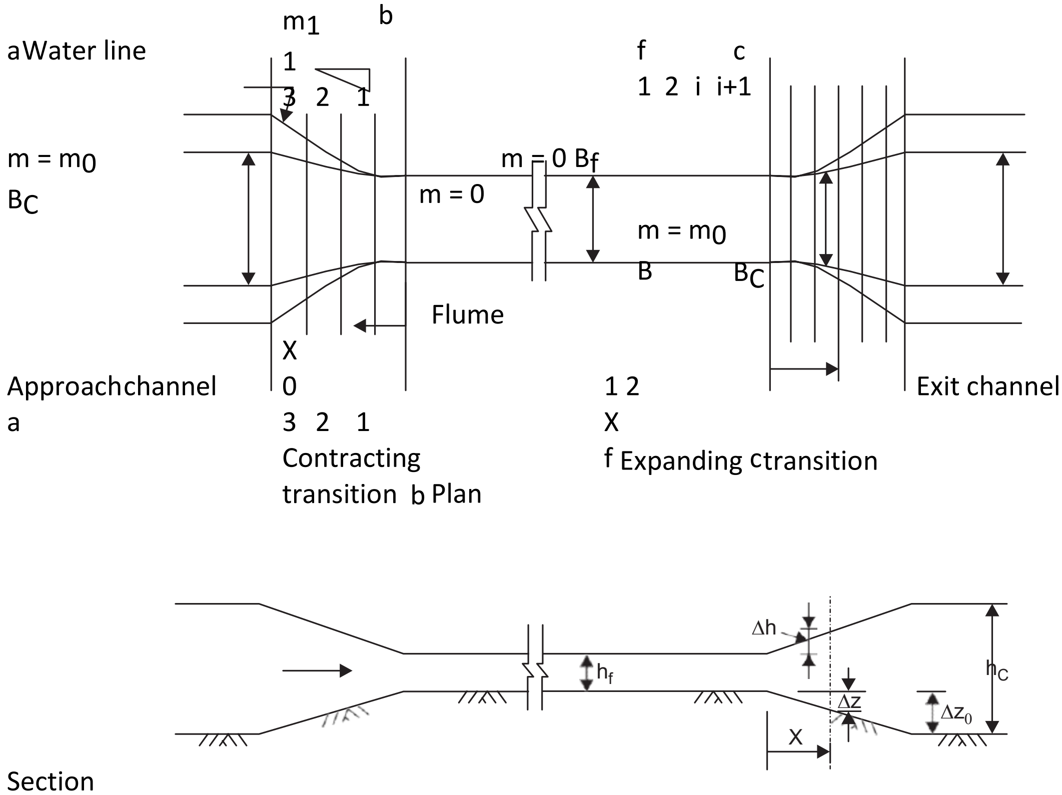


Fig. 11.10 Line sketch of canal trough and transitions

The flow rate Q is related to the depth of flow h and velocity of flow v in the transition as follows:

$Q = (B + mh)\, vh$ (11.7)

Applying Bernoulli's equation between the i^{th} and $(i + 1)^{th}$ sections (with flume bottom as the datum), one obtains

$$-\Delta z_i + (\Delta z_i + h_f + \Delta h_i) + \frac{v_i^2}{2g} = -\Delta z_{i+1} + (\Delta z_{i+1} + h_f + \Delta h_{i+1}) + \frac{v_{i+1}^2}{2g} + h_{L_{i,i+1}} \quad (11.8)$$

Here, h_f is the depth of flow in the flume and $h_{L_{i,\,i+1}}$ is the energy loss between the i^{th} and $(i + 1)^{th}$ sections. Further, Δz is considered as positive when the transition bed is lower than the flume bed, and the water surface elevation increment Δh (with respect to the water surface in the flume) is positive when the water surface in the transition is higher than the water surface in the flume (4). Equation (11.8) can, alternatively, be written as

$E_{i+1} = E_i + \Delta z_{i,\,i+1} - h_{L_{i,\,i+1}}$ (11.9)

where, $\Delta z_{i,\,i+1} = \Delta z_{i+1} - \Delta z_i$

and specific energy $E = \Delta z + h_f + \Delta h + \dfrac{v^2}{2g}$

Hinds (5) proposed the following equation for estimating the head loss through transitions:

$$h_{L_{i,i+1}} = K_h \frac{v_i^2 - v_{i+1}^2}{2g} = K_h \Delta h_v \quad (11.10)$$

On substituting Eq. (11.10) into Eq. (11.9) and simplifying, one gets,

$$\Delta z_{i+1} + h_f + \Delta h_{i+1} + \frac{v_{i+1}^2}{2g} = \Delta z_i + h_f + \Delta h_i + \frac{v_i^2}{2g} + \Delta z_{i+1} - \Delta z_i - K_h \Delta h_v$$

$$\Rightarrow \quad \Delta h_{i+1} - \Delta h_i = \Delta h_v (1 - K_h)$$

$$\text{or} \quad \Delta h_v = \frac{\Delta h_{i+1} - \Delta h_i}{1 - K_h} \quad (11.11)$$

Hence, from Eqs. (11.10) and (11.11), one obtains,

$$h_{L_{i,i+1}} = \frac{K_h (\Delta h_{i+1} - \Delta h_i)}{(1 - K_h)} \quad (11.12)$$

A good transition design would require proper selection of the transition geometry (or determination of suitable values for B, Δz, and m of Eqs. (11.4-11.6) which would yield minimumenergy loss consistent with the convenience of design and construction.

Following methods of design of transitions have been described here:

Hinds' method (5).

UPIRI method (6) which is commonly known as Mitra's method.

Vittal and Chiranjeevi's method (4).

Hinds' Method

Hinds (5) assumed a water surface profile,

$$\Delta h = f_4 (x) \quad (11.13)$$

in the transition as a compound curve consisting of two reverse parabolas with an inflexion point in the middle of the transition and which join the water surface at either end of the transition tangentially. The water surface profile equation [Eq. (11.13)] is, therefore, writtenas

$$\Delta h = C_1 x^2 \quad (11.14)$$

Here, C_1 is a coefficient to be determined from the coordinates of the junction of two parabolas, and x is to be measured from the transition end for the respective parabolas. Hinds (5) also assumed a linear rise (for contraction) or drop (for expansion) in the channel bed. Thus,

$$\Delta z = C_2 x \quad (11.15)$$

where, C_2 is a constant.

The transition is now divided into N sub-reaches by cross-sections 1-1, 2-2, etc., and an arbitrary set of values for m, lying between 0 and m_0 (i.e., $0 \le m \le m_0$), is assigned to these sections. Here, m_0 is the side slope of the exit (or approach) channel. Using Eqs. (11.14) and (11.15), one can compute Δh and Δz and, hence, the depth of flow (= $\Delta z + h_f + \Delta h$) for any cross- section of the transition. Using Eqs. (11.12) and (11.8), one can determine the flow velocity at the (i + 1)th section for known v_i, and substitution of v_{i+1} in Eq. (11.7) yields the bed width at the (i + 1)th section. These computations proceed from one end of the transition to the other end. If the resulting transition is not smooth and continuous, the computations are repeated with a new set of arbitrary values for m till a smooth and continuous bed width profile is obtained.

UPIRI Method

For channels of constant depth, it was assumed (6) that the rate of change of velocity per unit length of the transition should be constant throughout the transition length. This means,

$$(v_f - v_i)/x = (v_f - v_c)/L \quad (11.16)$$

Here, suffixes f and c are, respectively, for the flumed section and normal section of the channel, and x is the distance of the ith section of the transition from the flumed section. Thus, v_i is the velocity of flow at the chosen ith section. L is the length of the transition which was arbitrarily assumed as 2 ($B_c - B_f$). Since the depth h is constant,

$$B_f v_f = B_i v_i = B_c v_c = Q/h$$

Hence,

$$v_f = \frac{Q}{h} \times \frac{1}{B_f}$$

$$v_i = \frac{Q}{h} \times \frac{1}{B_i}$$

and

$$v_c = \frac{Q}{h} \times \frac{1}{B_c}$$

Therefore, using Eq. (11.16),

$$\frac{(Q/hB_f) - (Q/hB_i)}{x} = \frac{(Q/hB_f) - (Q/hB_c)}{L}$$

or

$$B_i = \frac{L B_c B_f}{L B_c - x(B_c - B_f)} \quad (11.17)$$

Equation (11.17) is the hyperbolic bed-line equation for constant depth and can be used for transition between rectangular flume and rectangular channel.

Vittal and Chiranjeevi's Method

Vittal and Chiranjeevi (4) examined the above two methods and offered the following comments:

Hinds' method involves a trial procedure which can be easily avoided if the values assigned to m follow a smooth and continuous function [Eq. (11.6)] instead of an arbitrary set of values for m. The function to be chosen should be such that the side slope varies gradually in that part of the transition where the velocities are higher and it varies rather fast in other part of the transition where the velocities are lower.

A smooth and continuous bed width profile alone is not sufficient to avoid separation and consequently the high head loss.

The total head loss through transition can be obtained by summing up all the head losses in the sub-reaches of the transition. Using Eq. (11.10), the total head loss H_L is given by

$$H_L = \sum h_{L_{i,i+1}} = K_h \sum \frac{v_i^2 - v_{i+1}^2}{2g} = K_h \frac{v_f^2 - v_c^2}{2g} \quad (11.18)$$

This means that the head loss in the transition depends only on the entrance and exit conditions, and is unaffected by the transition geometry. This, obviously, is not logical and is a limitation of Hinds' method.

Hinds' method first assumes free water surface and then computes the boundaries which would result in the assumed water surface. However, it is usually desirable to select the boundaries first and then compute the water surface profile.

UPIRI method (6) would require lowering of the flume bed below the channel bed to achieve constant depth. This may not be always practical. For example, in case of a cross-drainage structure, the flume bed level may have to be lowered even below the drainage HFL. As a result, the cross-drainage structure, which otherwise can be an aqueduct, may have to be designed as a siphon aqueduct which is relatively more expensive.

On the basis of theoretical and experimental investigations, Vittal and Chiranjeevi (4) developed a method for the design of an expanding transition. The guiding principles for this method were minimisation of the energy loss and consideration of flow separation in the expanding flow. The design equations of Vittal and Chiranjeevi (4) for the bed width and sideslope are as follows:

Transition bed width profile:

$$\frac{B - B_f}{B_c - B_f} = \frac{x}{L}\left[1 - \left(1 - \frac{x}{L}\right)^n\right] \quad (11.19)$$

where, $n = 0.80 - 0.26\, m_0^{1/2}$ (11.20)

and length of transition, $L = 2.35\,(B_c - B_f) + 1.65\, m_0\, h_c$ (11.21)

Side slopes, varying according to the equation,

$$\frac{m}{m_0} = 1 - \left(1 - \frac{x}{L}\right)^{1/2} \quad (11.22)$$

change gradually in the initial length of the transition where the flow velocity is high, and rapidly in the latter length of the transition where the velocity is low. For head loss computations, use of Hinds' equation, viz., Eq. (11.10) with $K_h = 0.3$ has been suggested.

This method of design of expanding transition is applicable to all three types of conditions, viz., constant depth, constant specific energy, and variable depth-variable specific energy. The profiles of the bed line and the side slope will be the same for all the three conditions. The bed of the expanding transition would always rise in the direction of flow for the constant depth scheme. On the other hand, constant specific energy and variable depth-variable specific energy schemes would always result in the falling bed transition.

For the constant specific energy condition,

$$h + Q^2 \;\ldots\; E \;\ldots\; h + Q^2$$

E_C = $_c$ 2 g $B_c^2 h_c^2$ i i $2\,g\,B_i^2\,h_i^2$

Therefore, one can determine h_i and hence v_i. The transition loss between successive sections can be determined from Eq. (11.10) with K_h = 0.3. The bed has to be lowered at successive sections just sufficient to provide for the transition loss [as can be seen from Eq. (11.9)] so that the specific energy remains constant throughout the transition. This means that,

$\Delta z_{i,\, i+1} = h_{L_{i,i+1}}$ (11.23)

Equations (11.19) to (11.23) along with Eq. (11.10) enable the design of an expanding transition as has been illustrated in Example 11.1.

Similarly, for the constant depth condition, $h_f = h_c = h_i$

Hence, one can calculate L, n, B_i and m_i for known value of x using Eqs. (11.19 – 11.22). Thereafter, one can compute the velocity v_i and the head loss $h_{L_{i,i+1}}$ using Eqs. (11.7 and 11.10). Using Eq. (11.9), one can estimate the drop (or rise) $\Delta z_{i,\, i+1}$ in the bed elevation at the known value of x from the flume end.

The method based on variable depth-variable specific energy scheme presupposes the value of Δz_0 (i.e., difference in elevations of canal bed and flume bed) and the transition bed slope is assumed to be constant in the entire length of the transition. Length of transition L, the width of transition B_i and side slope m_i at known value of x (from the flume end) are calculated using the relevant equations discussed above. The depth of flow h_i is determined by trial using energy equation. To start with, the head loss term (unknown since the depth and, hence, velocity are unknown) is neglected and the value of depth of flow h_i is determined from the energy equation. Using this value of the depth of flow one can determine the velocity and the head loss between the adjacent sections and using energy equation, one can compute new value of h_i which is acceptable if it does not differ from the just previous trial value. Otherwise, one should repeat the trial.

UPLIFT PRESSURE ON TROUGH

In case of siphon aqueducts, the roof of the culvert (or the underside of the trough) is subjected to uplift pressure because the downstream water level is higher than the lower surface of the culvert covering. This uplift pressure head at any point of the culvert covering is equal to the vertical distance between the hydraulic gradient line and the underside of the trough (or the roof of the culvert). The uplift pressure is maximum when the stream is carrying the highest flood. The worst condition for the design of roof covering of the culvert would occur when the highest flood condition coincide with no flow in the canal. Knowing the afflux and the downstream level, the hydraulic gradient line can be drawn as shown in Fig. 11.11. The maximum uplift pressure, obviously, occurs at the upstream end. The thickness of the culvert roof designed to support the load of canal water and its own dead weight would generally be adequate for uplift pressures too as part of the uplift pressure is counterbalanced by the weight of canal water.

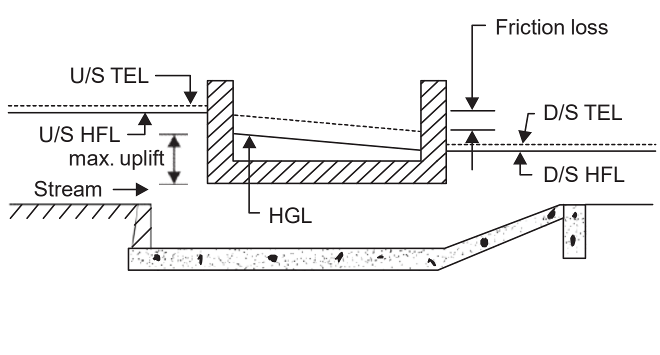


Fig. 11.11 Siphon aqueduct

UPLIFT PRESSURE ON CULVERT FLOOR

In siphon aqueducts, the culvert floor is subjected to uplift pressure due to: (i) the subsoil water (below the stream bed), and (ii) the water that seeps from the canal to the stream bed through the embankment (Fig. 11.12).

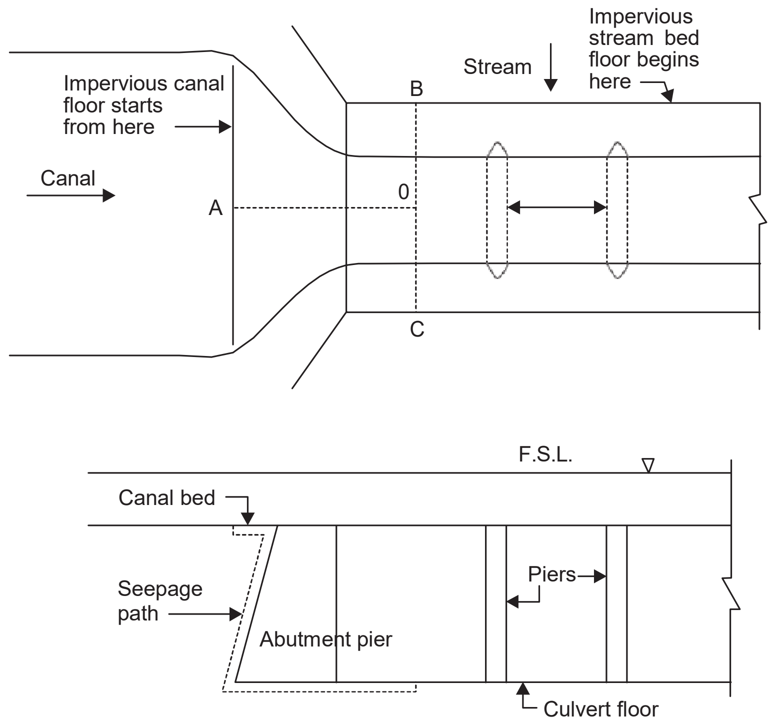


Fig. 11.12 Assumed path of seepage from canal to stream

For some part of a year, the water table below the stream bed may rise up to the bed level itself. This will exert uplift pressure on the floor of the culvert. The maximum uplift will occur when the stream is dry and will be equal to the difference of levels of the stream bed and bottom of the culvert floor.

As shown in Fig. 11.12, the canal water seeps to the stream bed and reappears at B and C on both sides of the culvert floor. This seepage is, obviously, due to the difference of head between the canal water level and stream water level. The maximum value of the uplift pressure on the culvert floor due to this seepage will occur when the canal is running at FSL and the stream is dry. This seepage flow is three-dimensional in nature and can only be approximately solved by numerical methods or model studies. However, for relatively small structures, the following simple method based on Bligh's creep theory can be used for the design (3). The method can also be used in the preliminary design of major important structures which should, however, be checked by model studies.

Measure the length of the seepage path AOB or AOC which, in fact, will be equal to the length of the seepage path, shown with dotted lines in elevation, and the distance OB (or OC). But, for further simplifying the calculation, the seepage length between A and O is taken as

equal to the length of impervious concrete floor on the canal bed up to the flumed end and one- half of the barrel span. If the seepage head, i.e., the difference between the canal FSL and level of the bottom of the culvert floor is H, and the total seepage length is l then the residual head at O, H_0 is equal to $H((l-l_l)/l)$ in which, l_1 is the length of seepage path between A and O, i.e., from the start of the impervious canal floor to the centre of the bottom of the floor of the first barrel. This residual head H_0 and the uplift due to subsoil water are to be counterbalanced by the weight of the culvert floor and, hence, the thickness of the culvert floor can be calculated. In these calculations, it is assumed that the barrels are dry and free of any sediment deposit so that the structure is safe even under the worst condition. However, the resulting floor thickness
will be excessive and uneconomical. Besides, higher floor thickness further increases the static uplift pressures. Therefore, the uplift pressure is transferred to the piers by utilising the bending strength of the floor. With such a provision, a floor of reasonable thickness (say, around 0.50 m) would be adequate. Sometimes, inverted arches are also used.

When uplift pressures on the culvert floor are excessive, some additional measures are taken to reduce the uplift. A simple measure would be to extend the impervious floor of the canal on both sides of the trough. Alternatively, the seepage water is made to emerge into the barrel by providing a number of openings (known as relief holes) in the culvert floor. Relief holes are in the form of pipes embedded in the barrel floor. However, the seeping water may bring with it the soil particles also resulting in subsidence of the floor in course of time. This can be prevented by providing inverted filter just below the culvert floor. This inverted filter would consist of one or more pervious layers of soil (inside the embedded pipes serving as relief holes) such that the permeability increases in the upward direction. However, the voids of these pervious layers are not large enough to permit the movement of subsoil particles with the seepage water. Terzaghi's criteria are used for the design of different layers of the inverted filter. These criteria are as follows:

$$\frac{D_{15} \text{ of filter layer}}{D_{15} \text{ of protected soil}} ? 4$$

and

$$\frac{D_{15} \text{ of filter layer}}{D_{85} \text{ of protected soil}} \leq 4$$

When the canal is dry, there exists a possibility of sediment-laden stream water entering the filter layers through the holes in the culvert floor and, thus, filling the filter voids with sediment. This can be prevented by providing upward-opening flap valves on the top of the relief holes.

MISCELLANEOUS DETAILS

The exit gradient of the seepage water at the downstream end of the culvert floor must be within permissible limits. Some nominal protection in the form of loose stone may also be provided on the upstream as well as the downstream of the impervious floor.

The foundation of any cross-drainage structure should meet the requirement of the bearing capacity of the soil under optimum loads, seismic effects, anticipated scour, and settlement.

To facilitate drainage of the backfills due to rise in water table or otherwise, small openings are provided in abutments, wing walls, and return walls. These openings are called weep holes.

In big siphons, stop log grooves on the sides of the upstream entrance and downstream outfalls should be provided so as to isolate one or more barrels for periodic inspections, repairs, and maintenance.

Trash racks are desirable at the entrance of the siphon where large quantity of floating material is expected in the channel water. These racks are generally made in panels for convenient handling. These trash racks are usually inclined at a slope of 1(H) : 4(V).

Example 11.1 Design a suitable cross-drainage structure for the following data: Discharge in the canal, = 357.0 m^3/s

Bed width of the canal, B_c = 23.0 m

Side slope of the canal, m_0 = 2.0

Bed level of the canal,= 267.0 m

Depth of flow in the canal, h_c= 6.7 m Bed width of the flume, B_f = 15.0 m High flood discharge of the stream = 500 m^3/s

High flood level of the stream = 268.0 m Bed level of the stream = 265.0 m

Solution: Using Eq. (11.21), the length of expansion transition,

$L = 2.35 (B_c - B_f) + 1.65\, m_0 h_c$

$= 2.35 (23 - 15) + 1.65 \times 2 \times 6.7 = 40.91$

≈ 41 m

Let the transition be divided into 9 sub-reaches of 4.0 m each and the remaining 5 m length be considered as the tenth sub-reach at the end.

Bed width, b is calculated using Eqs. (11.19) and (11.20).

$n = 0.8 - 0.26\, m_0^{1/2}$

$= 0.8 - 0.26 (2)^{1/2} = 0.432$

$$\frac{B - B_f}{B_c - B_f} = \frac{x}{L}\left[1 - \left(1 - \frac{x}{L}\right)^n\right]$$

or $$\frac{B - 15}{23 - 15} = \frac{x}{41}\left[1 - \left(1 - \frac{x}{41}\right)^{0.432}\right]$$

or $$B = 15 + \frac{8x}{41}\left[1 - \left(1 - \frac{x}{41}\right)^{0.432}\right]$$

For example, at section 1-1,

$$B_{1-1} = 15 + \frac{8 \times 4}{41}\left[1 - \left(1 - \frac{4}{41}\right)^{0.432}\right]$$

= 15.034 m

Values of B for different values of x have been similarly calculated and are as shown in col. 3 of Table 11.5.

Side slopes at various sections are calculated using Eq. (11.22). Thus

$$\frac{m}{m_0} = 1 - \left(1 - \frac{x}{L}\right)^{1/2}$$

or $$m = 2\left[1 - \left(1 - \frac{x}{41}\right)^{1/2}\right]$$

For example, at section 1-1,

$$m_{1-1} = 2\left[1 - \left(1 - \frac{4}{41}\right)^{1/2}\right] = 0.100$$

Values of m for different values of x have been similarly calculated and are tabulated in col. 4 of Table 11.5.

For constant specific energy condition:

$$E_f = E_c = h_c + \frac{v_c^2}{2g}$$

and $v_c = 357/[6.7\,(23 + 2 \times 6.7)] = 1.464$ m/s

$$\therefore \quad E_f = E_c = 6.7 + \frac{(1.464)^2}{2 \times 9.81} = 6.809 \text{ m}$$

$$h_f + \frac{Q^2}{2gB_f^2 h_f^2} = 6.809$$

or $$h_f + \frac{(357)^2}{2g(15)^2 h_f^2} = 6.809$$

Solving for h_f, one obtains $h_f = 6.006$ m

Similarly, $$E_{1-1} = h_{1-1} + \frac{Q^2}{2g(b_{1-1} + m_{1-1} h_{1-1})^2 h_{1-1}^2} = E_c = 6.809 \text{ m}$$

$$\therefore \quad h_{1-1} + \frac{(357)^2}{2 \times 9.81\,(15.034 + 0.1\,h_{1-1})^2 h_{1-1}^2} = 6.809$$

On solving this equation, $h_{1-1} = 6.088$ m

Further, $$v_f = \frac{357}{15 \times 6.006} = 3.963 \text{ m/s}$$

and $$v_{1-1} = \frac{357}{(15.034 + 0.1 \times 6.088)\,6.088} = 3.749 \text{ m/s}$$

Using Eq. (11.10), $$h_{L_{f,1-1}} = 0.3 \times \frac{1}{2 \times 9.81}$$

$(3.963^2 - 3.749^2) = 0.025$ m

Using Eq. (11.23), $\Delta z_{f,1-1} = 0.025$ m

The positive value of Δz indicates fall in the bed elevation with respect to the flume bed. Values of Δz for other sections can be similarly obtained. The values of Δz and h have been tabulated in cols. 5 and 6 of Table 11.5.

Table 11.5 Designed parameters of expanding transition (Ex. 11.1)

Section	x	Vittal and Chiranjeevi method				Hinds' method					
		B (m)	m	z	h (m)	Δh (m)	h (m)	z	v (m/s)	m	B (m)
1	2	3	4	5	6	7	8	9	10	11	12
f-f	0.000	15.000	0.000	0.000	6.006	0.000	6.006	0.000	3.963	0.000	14.999
1	4.000	15.034	0.100	0.025	6.088	0.013	6.019	0.020	3.793	0.100	15.037
2	8.000	15.140	0.206	0.050	6.173	0.053	6.059	0.040	3.614	0.200	15.090
3	12.000	15.325	0.318	0.074	6.255	0.119	6.125	0.061	3.427	0.300	15.172
4	16.000	15.601	0.438	0.096	6.327	0.211	6.217	0.081	3.228	0.400	15.302
5	20.000	15.980	0.569	0.117	6.408	0.330	6.336	0.101	3.017	0.500	15.510
6	24.000	16.482	0.712	0.135	6.463	0.239	6.461	0.121	2.789	0.600	15.935
7	28.000	17.138	0.874	0.152	6.517	0.140	6.560	0.141	2.541	0.800	16.168
8	32.000	18.002	1.063	0.168	6.572	0.067	6.633	0.162	2.266	1.000	17.115
9	36.000	19.196	1.302	0.183	6.613	0.021	6.679	0.182	1.953	1.200	19.350
c-c	41.000	23.000	2.000	0.203	6.700	0.000	6.700	0.207	1.472	2.000	22.795

Steps for the design of the expanding transition using the Hinds' method would be as follows:

Consider the same length of the transition as obtained earlier i.e., 41 m. One could, alternatively, select a suitable length on the basis of minimum splay consideration. Consider also the same condition of constant specific energy so that the velocity and the depth at the flume end are the same as obtained earlier.

Using Eq. (11.14) at the mid-section (i.e., = 41/2 = 20.5 m from the transition end) of the transition where,

$\Delta h = (6.7 - 6.006)/2 = 0.347$ m

$\therefore \quad C_1 = \Delta h/x^2 = 0.347/(20.5)^2 = 8.26 \times 10^{-4}$

Thus, Eq. (11.14) becomes $\quad \Delta h = 8.26 \times 10^{-4} x^2$

For section 3-3, x = 12 m (from the flume end)

$\therefore \quad \Delta h = 0.119$ m

$\therefore \quad h = 6.006 + 0.119 = 6.125$ m

Similarly, for section 8-8, x = 9 m (from the canal end)

$\therefore \quad \Delta h = 0.067$ m

$\therefore \quad h = 6.700 + 0.067 = 6.633$ m

The values of Δh and h for other sections of the chosen sub-reaches of the transition are computed similarly and the values have been tabulated in cols. 7 and 8 of Table 11.5.

Head loss in transition, $h_{L,\, f-c}$ is given by Eq. (11.10),

$h_{L,f-c} = 0.3\ (v_f^2 - v_c^2)/2g$

$= 0.3\ [(3.963)^2 - (1.464)^2]/(2 \times 9.81) = 0.207$ m

For constant specific energy, the change in bed elevation between the two ends of the transition would be equal to the head loss between those ends. Therefore,

$\Delta z_{f-c} = 0.207$ m

This drop (transition being an expanding one) of 0.207 m is assumed to be linear and the values are listed in col. 9 of Table 11.5. Writing Eq. (11.10) between flume and any section,

$$h_{L,f-i} = 0.3\,(v_f^2 - v_i^2)/2g$$

$$\therefore \quad v_i^2 = \sqrt{(15.705 - 65.4 \times h_{L,f-i})}$$

Since $h_{L,\,f-i}$ is equal to Δz_{f-i}, one can compute v_i. The computed values are listed in col. 10 of Table 11.5. The transition width, B at any section can now be computed, using Eq.
(11.7), if the side slopes at various sections of the transition are known. These side slopes are to be assumed arbitrarily so that the resulting profile of the transition is smooth and also feasible. For the chosen values of side slope as listed in col. 11 of Table 11.5, the computed values of the transition widths at various sections are as shown in col. 12 of Table 11.5. One needs to adjust the width suitably or try another trial with other set of the values of the side slopes.

Design of Contracting Transition:

The bed-line profile of the contracting transition can be obtained from Eq. (11.3). Adopting a value of m_1 equal to 4, Eq. (11.3) becomes

$$\left(\frac{x}{16}\right)^2 + \left(\frac{y}{4}\right)^2 = 1$$

Length of transition $= 0.5\, m_1(B_c - B_f)$

$= 0.5 \times 4(23 - 25) = 16$ m

Dividing the contracting transition reach into eight sub-reaches, one can determine the values of y and, hence, the bed width ($= B_c - 2y$) for known values of x measured from the flume end of the transition.

The side slopes and bed elevation may be varied linearly. The computed values of bed width and side slopes are shown in Table 11.6.

Table 11.6 Values of bed width and side slopes of contracting transition (Example 11.1)

x, (m)	0	2	4	6	8	10	12	14	16
y, (m)	4.0	3.97	3.87	3.71	3.46	3.12	2.65	1.94	0.0
$B = B_c - 2y$, (m)	15.00	15.06	15.25	15.58	16.08	16.76	17.70	19.12	23.00
m	0	0.25	0.5	0.75	1.0	1.25	1.5	1.75	2.00

The trough can be divided into three compartments each of width 5.0 m, separated by two intermediate walls 0.30 m thick. The two side walls may be kept 0.60 m thick and 7.3 m high so that a freeboard of 0.60 m is available over 6.7 m depth of flow. For further illustration, let the thickness of the bottom slab be 0.60 m. These dimensions yield the overall width of the trough (i.e., the length of siphon barrel) equal to 15 + (0.3 × 2) + (0.6 × 2) = 16.8 m.

Waterway of the Stream:

Since the bed level of the flumed canal (i.e., 267.203) is below the HFL of the stream (i.e., 268.00), a siphon aqueduct will be suitable.

Lacey's regime perimeter, $P = 4.75\sqrt{Q}$

$= 4.75\sqrt{500} = 106.2$ m

If the clear span of barrel is fixed as 8 m, the pier thickness is obtained as

$$0.55\sqrt{\text{barrel span in metres}} = 0.55\sqrt{8} = 1.56 \text{ m} = 1.6 \text{ m (say)}$$

The overall waterway for 12 spans of 8 m each and 1.6 m pier thickness would be

$$12 \times 8 + 11 \times 1.6 = 113.6 \text{ m}$$

The clear waterway is 96 m.

For velocity of flow in the barrels equal to 2.5 m/s, the height of barrel should be

$$\frac{500}{96 \times 2.5} = 2.083 \text{ m}$$

Providing a height of 2.0 m for barrels, the flow velocity in the barrels is

$$\frac{500}{96 \times 2.0} = 2.60 \text{ m/s}$$

For these barrels, hydraulic radius, $R = \dfrac{8 \times 2}{2\,(8 + 2)} = 0.8$ m

The loss of head through siphon barrels (16.8 m long) can be obtained from Eq. (11.2). Neglecting approach velocity,

$$h_L = \left(1 + f_1 + f_2 \frac{L}{R}\right)\frac{V^2}{2g}$$

where $f_1 = 0.505$, and $f_2 = a\left(1 + \dfrac{b}{R}\right)$

For smooth cement plaster surface, a = 0.00316 and b = 0.03

$$\therefore \quad f_2 = 0.00316\left(1 + \frac{0.03}{0.8}\right) = 0.00328$$

$$\therefore \quad h_L = \left(1 + 0.505 + 0.00328\,\frac{16.8}{0.8}\right)\frac{(2.6)^2}{2 \times 9.81} = 0.54 \text{ m}$$

$\therefore$ Upstream HFL = 268 + 0.54 = 268.54 m

Uplift pressure on the barrel roof (or flume trough):

Consider the downstream end of the flume and transition designed using Vittal and Chiranjeevi's method.

For the chosen thickness of the trough slab as 0.6 m and the computed value of Δz as 0.203 m, R.L. of the bottom of the trough (i.e., culvert roof) slab

= 267.0 + 0.203 – 0.6

= 266.603 m

$$\text{Loss of head at the entry of the barrel} = 0.505\,\frac{V^2}{2g} = 0.505\,\frac{(2.6)^2}{2g} = 0.174 \text{ m}$$

$$\text{and velocity head in the barrel} = \frac{V^2}{2g} = \frac{(2.6)^2}{2g} = 0.345 \text{ m}$$

Part of the hydraulic head available at the upstream end of the barrel, Fig. 11.11, is utilized in meeting the entry (to the barrel) loss and developing the velocity head in the barrel. Therefore, maximum uplift (at the upstream end) pressure on the trough slab (Fig. 11.11)
= HFL – Entry loss – velocity head in the barrel
– RL of the bottom of the trough
= 268.54 – 0.174 – 0.345 – 266.603
= 1.418 m
Uplift pressure on the floor of the barrel : RL of barrel floor = 266.03 – 2.0
= 264.603 m
Let the floor thickness of the barrel be 1.5 m
 RL of the bottom of the barrel floor = 264.603 – 1.5 = 263.103 m
 Static pressure on the barrel floor = 265 – 263.103 = 1.897 m
The culvert floor should extend toward the upstream by a distance equal to the difference between HFL and the culvert floor level, i.e., 268 – 264.603 = 3.397 m 3.5 m (say)
Length of upstream contracting transition (of canal) = 16 m Half of the barrel span = 4 m
End of the culvert floor from the centre of the barrel = 3.5 + (16.8/2) = 11.9 m Total seepage head (at the upstream end of the culvert floor)
= FSL of canal (in the flumed portion) – Bed level of the stream
= (267.203 + 6.006) – 265.0 = 8.209 m
Therefore, residual seepage head at the centre of the barrel (calculated approximately)
= 8.209 [31.9 (16 4)]
(16 4 11.9)
= 8.209 11.9 = 3.06 m
31.9
Therefore, total uplift at the bottom of the culvert floor = 1.897 + 3.06 = 4.957 m
As in case of other hydraulic structures, suitable wing connections and protection works on both the upstream and downstream sides of the canal and the stream at the site of the structure are to be provided.

3.1.3 Hydrology and Agro- meteorology

<u>Introduction</u>

General introduction

Hydrology is derived from two words: hydro and logos. ‘Hydro’ means water and ‘logos’ means study. Simply, Hydrology is defined as the study of water.
Hydrology is the science which deals with the origin, distribution and circulation of water in different forms in land phases and atmosphere.

Broad definition of Hydrology given by US National Research Council
Hydrology is the science that treats the waters of the Earth, their occurrence, circulation and distribution, their chemical and physical properties, and their reaction with the environment, including their relations to living things.

Interdisciplinary subject: As an earth science, Hydrology is connected to several subjects. These include:
Meteorology: for understanding precipitation and evaporation process
Soil science: for understanding infiltration
Geology: for understanding groundwater flow
Geomorphology: for understanding surface runoff

Hydraulics: for understanding stream flow
Physics, Chemistry, Biology, Math, statistics: to formulate and understand the subject

Division of hydrology
Scientific hydrology: deals with hydrological processes from the view point of natural processes
Applied or engineering hydrology: deals with engineering applications of hydrology

Scope of hydrology
Estimation of water resources
Study of processes like precipitation, evaporation, infiltration and runoff and their interaction
Study of problems like floods and droughts
Understanding the properties of water in nature

Things to be considered for planning and design of water resources projects
Maximum flows which are expected to occur at a place
Minimum flows which can occur during any dry period
Minimum reservoir capacity to be fixed to meet all water demands from a multipurpose reservoir
Possible regulation of floods at the downstream reaches once a hydraulic structure is erected
Possible supply of water from a river to meet demands for water resources projects
Environmental impacts of a hydraulic structure
Study of groundwater potential and its use
Watershed/catchment/drainage basin

Watershed is the area of land draining into a stream at a given location. Divide is a line which separates catchment from its neighbouring catchments. For delineating basin, we need topographic map. The map shows changes in elevation by using contour lines.
Features of contour
Uphill: contour with higher elevation
Hill: circular contour, ridge: highest point
Saddle: mountain pass
Valley: V or U shaped with the point of the V/U being the upstream end
Close together contours: steep slope
Widely spaced contour: level ground

Basin delineation procedure on topo map
Mark the outlet point
Mark the highest point around the river
Start from the outlet and draw line perpendicular to the contours in such a way that the line passes from the highest point (ridge)
Continue to the opposite side of the watercourse, finally ending to the outlet.

Finding area of watershed/basin
Method 1: Use planimeter around the boundary Method 2: Trace the basin and count area manually Method 3: Use of GIS package

Applications of Hydrology in Engineering
Correct assessment of flows for hydropower, irrigation, drainage and water supply projects.
Determination of maximum expected flow at dam, reservoir, spillway, bridges, culverts and city drainage system.
Determination of minimum reservoir capacity sufficient to meet the hydropower, irrigation and water supply demands.

Estimation of the total volume of water that may be available from a drainage basin over a longperiod
Flood control: statistical analysis of probable frequency of floods, estimation of design flood, andflood forecasting.
computation of water surface profile for various rates of flow for navigation
Control of erosion to minimize sedimentation of reservoirs.
Reduction of stream pollution

Hydrological cycle

The endless circulation of water between the earth and its atmosphere is called hydrological cycle. Hydrological cycle is the most fundamental principle of hydrology. The cycle extends its scope from 15 km up into the atmosphere from the earth's surface to about 1km below the earth's crust through a maze of paths. It is fueled by solar energy and driven by gravity force.

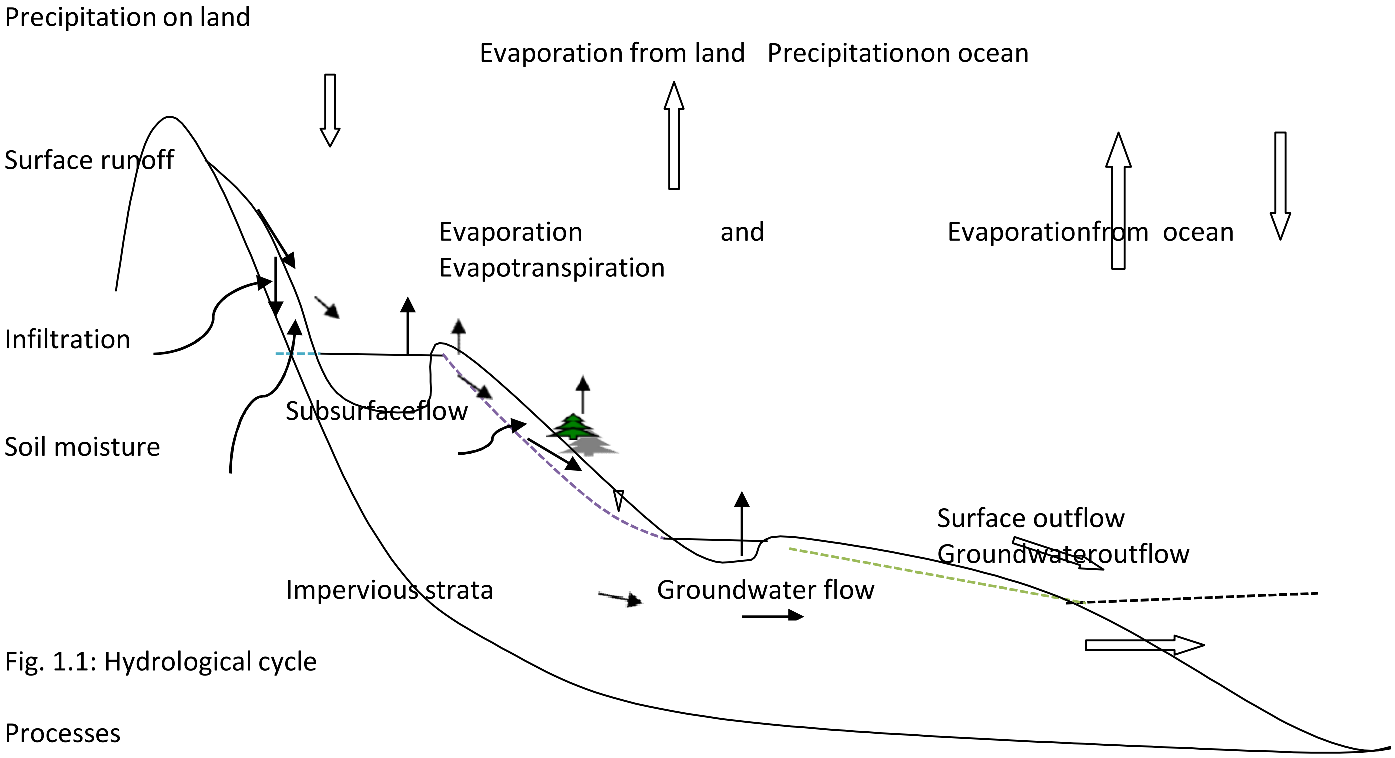


Fig. 1.1: Hydrological cycle

Processes

Evaporation: Water is evaporated from the oceans and land surfaces to become part of theatmosphere.

Precipitation: Water vapour is transported and lifted in the atmosphere until it condenses andprecipitates (falls in the form of solid or liquid) on the land or the oceans.

Interception: Part of precipitation is intercepted by vegetation and trees.

Infiltration: Part of precipitation infiltrates into the soil.

Surface runoff (Overland flow): The fallen precipitation flows over the land surface before reaching the channel

Evaporation and Transpiration: Much of the intercepted water and surface runoff returns to the atmosphere through evaporation. Part of the infiltrated water is available to the roots of the trees and returns to the atmosphere through plant leaves by transpiration.

Subsurface runoff (Interflow): The infiltrated water flows laterally through the unsaturated soil to the stream channel.

Deep percolation: The water from the soil moisture zone percolates deeper to recharge ground water.

Ground water flow (Base flow): The flow takes place from the saturated groundwater zone to the streams.

Final output: Streamflow

The part of precipitation that reaches the stream through different paths above and below the earth surface is called runoff. Once it enters the channel, the runoff is called streamflow.

Finally the precipitated water flows out into the sea which it will eventually evaporate once again and the hydrological cycle continues.

Water budget or water balance equation

The water balance equation is the statement of the law of conservation of mass. Water balance is the balance of input and output of water within a given area taking into account net changes of storage.

Change in storage = Inflows-Outflows

$$\frac{d}{dt}(Storage) = Inflows - Outflows$$

It is also called continuity equation or conservation equation.

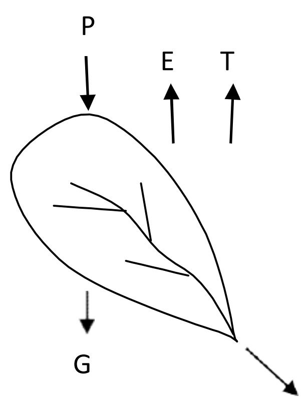


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Fig. 1.2: Various components of water balance in a basin

General water budget equation in hydrology for time interval Δt

$P - (R + G + E + T) = \Delta S$

P= precipitation
R = Surface runoff
G = Net groundwater flow out of the catchment E = Evaporation
T = Transpiration
ΔS = change in storage (take + for increase in storage, and – for decrease in storage)

All the terms in the equation have the dimensions of either volume or depth. Conversion of unit given basin area: Volume = Depth x basin area Conversion to volume given flow rate: Volume = flow rate x time duration

In case of other inflow besides precipitation, the water balance equation is

$(P + I) - (R + G + E + T) = \Delta S$ where I= other inflow

For long term, e.g. annual water balance, change in storage is zero. The general water balance equation is: Precipitation-Runoff=Evaporation

Significance of water balance equation

The water balance equation is useful to assess the various components of the hydrological processes for a certain time interval. The input to the system (basin) is precipitation, and the outputs from the system are surface runoff, evaporation, transpiration and groundwater flow. Storage component is the water stored within the basin. The assessment of runoff using water balance equation is useful for water resources projects, such as irrigation, water supply, flood control, pollution control etc. The equation is also useful for estimating the change in storage in a reservoir and estimating losses from precipitation.

History of hydrological development in Nepal

History of hydrological development in Nepal is not very long. Different activities in the development ofhydrology of Nepal are summarized below.
Preliminary works in the period of 1940-1960
Starting of Hydrological studies in Nepal after the Government of India initiated Koshi project inlate 1940s.
Establishment of Hydrological stations on Koshi at Barahachhetra, Sunkoshi at Kampughat andTamur at Mulghat in 1947.
Establishment of meteorological observations stations in 1956 with the support of theGovernment of India.
Establishment of Department of Hydrology and Meteorology
Nepal started hydrological and meteorological activities in an organized way in 1962 from theKarnali basin. The activities were initiated as a section under the Department of Electricity.
Establishment of the Department of Hydrology and Meteorology (DHM) under the ministry ofWater and Power in 1966.
Publication of hydro-meteorological data from 1966.
Merging of the DHM with the Department of Irrigation in 1972.
Separation of DHM in 1988 from the irrigation.
Main responsibilities of DHM: collection, analysis, processing, dissemination of hydrometrorological data; meteorological and hydrological forecasting; research work on hydrology and meteorology
Starting of Nationwide hydro-meteorolgical data management project in 1993.
From recent years, use of modern technology for data collection such as wireless communication, satellite data receiving system, receiving data through internet using CDMA

Station network
Koshi, Gandaki, Karnali, Mahakali, Bagmati, Kamala, Kankai, Babai, West Rapti are major basins of Nepal. The topography of Nepal has a major role in determining the hydrological network. The hydrological network in Nepal is very poor in headwater region, fair in mountainous region and again poor in the Terai region. At present, DHM maintains 154 hydrological stations and 337 precipitation stations all over Nepal.

Acts, plan
Water Resource Act (1992): act enacted for rational utilization, conservation, management and development of the water resources of Nepal
Water Resources Strategy (WRS, 2001): strategies formulated to improve the living standard of people through the water resource development
National water plan (NWP, 2005): In order to implement the activities identified by the WRS, the Water and Energy Commission Secretariat (WECS) formulated National Water Plan (NWP) in 2002, which was approved in 2005. The NWP is a framework to guide, in an integrated and comprehensive manner, all stakeholders for developing and managing water resources and water services.

Precipitation

Precipitation
Precipitation is any form of solid or liquid water that falls from the atmosphere to the Earth's surface. Forms of precipitation
Drizzle: water droplets, low intensity, 0.1 – 0.5 mm
Rain: water droplets, higher intensity, > 0.5 mm
Glaze: ice coatings formed by freezing rain

Sleet: ice grains formed due to freezing temperature while falling
Snow: ice crystals, hexagonal
Hail: ellipsoid ice balls, 5 to 125 mm
Dew: during clear nights, when the surface of the object on the earth cools due to radiation, the moisture present in the atmosphere condenses on the surface of these objects forming water droplets called dew.

Requirements for precipitation formation
Uplift of air mass into atmosphere
A gradient in temperature in the atmospheric column
- Decrease of air temperature with altitude influences the amount of moisture that can be held as watervapor in air.
Water vapor in the atmosphere and saturation as temperature changes
In higher parts of the atmosphere (colder), there is less ability to hold water vapor (decrease in vaporpressure).
– Implies that colder air holds less water vapor, leads to saturated conditions (moist air) and promotescondensation.
Presence of nuclei (salt, dust, and clay around 1 to 10 µm in diameter, called aerosols) around whichcondensation of vapor takes place.
Precipitation product must reach the ground in some form.

Mechanism of precipitation formation
Water vapor rises in the atmosphere and cools.
Water droplets in clouds are formed by nucleation of vapor on aerosols.
Droplets increase in size by condensation.
Droplets (~0.1mm) become heavy enough to fall.
Many of the falling droplets decrease in size by evaporation, which are again carried upwards in the cloud.
Some of the falling droplets increase in size by impact and aggregation. Some of the larger drops (3-5mm) may break up into smaller raindrops and droplets. The droplets may be again carried upwards in cloud.
When the diameter of droplets becomes 0.1-3mm, they start falling.

Front
A front is the interface between two distinct air masses.
Types of precipitation based on lifting mechanism

Convective

Unequal heating at the surface of the earth is the main cause of convection. In summer days air in contact with the surface of the earth gets heated up, expands and rises due to lesser density. Surrounding cold air rushes to replace it and in turn gets heated up and rises thus setting up a convective cell. The warm air continues to rise and undergoes condensation. The condensation releases latent heat of vaporization, which helps to move the air mass up. Depending on the moisture content, cooling and other factors, the precipitation intensity varies from light showers to cloud bursts. Sometimes upward wind currents exceeding 150 kmph freezes the raindrops to form hail.

Orographic

Lifting of air mass over a mountain barrier is called orographic lifting. Dynamic cooling takes place causing heavy precipitation on the windward side and light on leeward side. Orographic precipitation gives medium to high intensity rainfall and continues for longer duration.

Cyclonic

A cyclone is a low pressure region surrounded by a larger high pressure area. The cyclone center is calledeye, which is a calm area. This zone is surrounded by strong wind zone. The pressure decreases towards eye.

When the low pressure occurs in an area, especially over large water bodies, air from the surroundings rushes, causing the air at low pressure zone to lift. The system derives its energy from sea vapor. Once thecyclone crosses over to the land, the energy source is cutoff, it becomes weak and disappears quickly. Therainfall is normally heavy in the entire zone travelled by a cyclone.

An anticyclone is an area of high pressure in which wind tends to blow spirally outward in clockwise direction in the northern hemisphere and anticlockwise in the southern hemisphere. Weather is usually calm and such anticyclones are not associated with rain.

Rainfall measurement

Precipitation is measured as depth of water equivalent from all forms that would accumulate on a horizontal surface if there are no losses.

Unit: mm or inch

Methods of precipitation measurement

Rain gauge

Radar

Satellite

Types of rain gauge

Non-recording gauge

The gauge which is read manually is called non-recording gauge. It does not record rain itself, but simply collects. It consists of collector above funnel leading into receiving vessel. The rainfall collected in the vessel is measured by a graduated measuring cylinder or dipstick to give depth of rainfall.

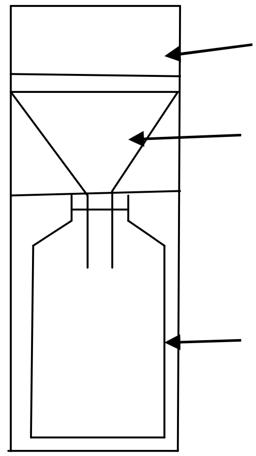


Fig. 2.1: Non-recording gauge

Recording gauge

The gauge which records the depth of rainfall automatically is called recording gauge. Rainfall intensity, duration and depth can easily be obtained from recording gauge. There are three types of recording gauge in general use.

Tipping bucket: Tipping bucket type gauge operates with a pair of buckets. When the rainfall first fillsone bucket, it tips and brings the other one in position. The flip-flop motion of the tipping buckets is transmitted to the recording device (clock-driven drum chart) and provides a measure of rainfall intensity.Alternatively, the tipping mechanism is used to actuate electric circuit which records the number of tips during rain. Usually one tipping is equal to 0.25mm of rain. The instrument is suitable for digital data.

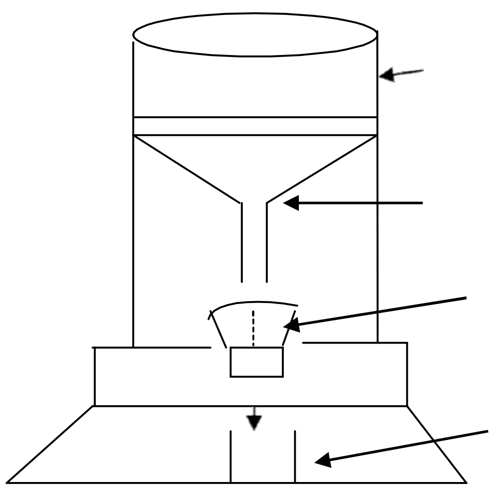


Fig. 2.2: Tipping bucket rain gauge

Weighing bucket: In this gauge, rainfall is collected in bucket which rests on a weighing scale with a spring mechanism. For recording the rainfall, mechanical lever arm of the balance is connected with a penwhich touches a clock mounted drum with a graph paper. The record shows accumulation of rainfall over time.

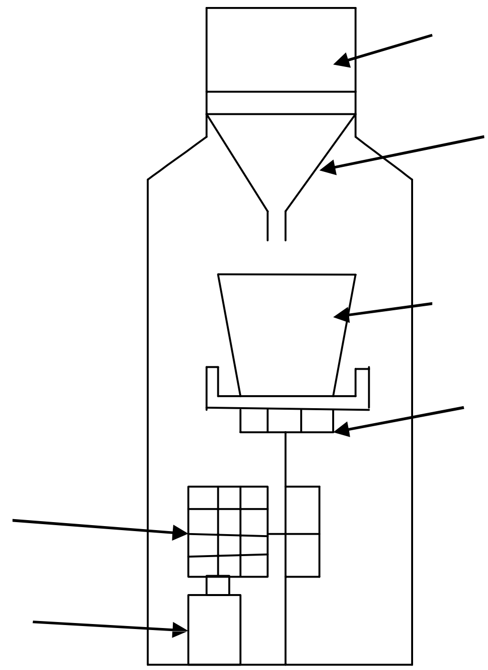


Fig. 2.3: Weighing bucket rain gauge

Float type (Syphon) gauge: This type of gauge has a chamber containing a float. With the increase in rainwater in the chamber, the float rises. Vertical movement of the float is translated into movement of a pen on a chart, which is mounted on a mechanical clock. A syphon arrangement empties the float chamber when the float has reached the pre-set maximum level. Then the pen comes back to original zeroposition showing vertical line on the graph. If there is no rainfall, the pen moves horizontally. Each syphonic action measures certain amount of rainfall, e.g. 10mm. This instrument provides mass curve of rainfall.

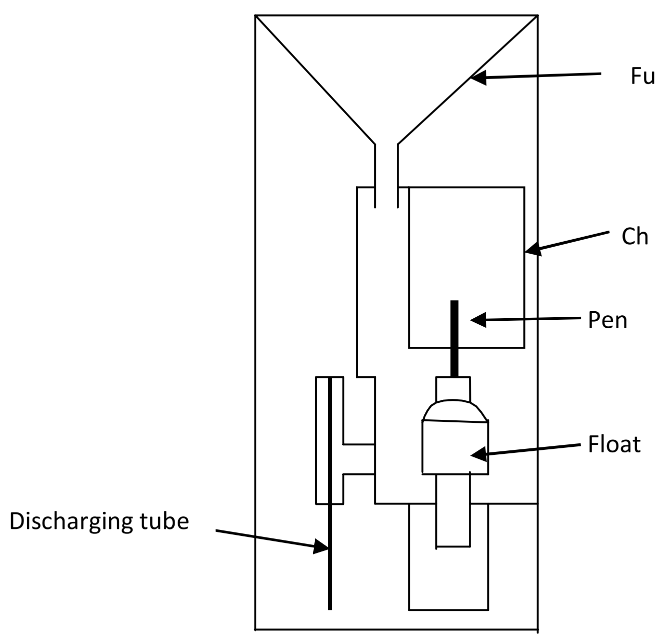


Fig. 2.4: Float type rain gauge

Error in measurement
Instrumental error
Human error
Wind error
Evaporation error
Wetting error
Splashing error

Telemetry

Telemetry is a system of transmitting rainfall data to a base station by means of electronic units connected to recording gauges. This system is very useful for mountainous and inaccessible areas. Telemetry is used for real time transmission of data.

Design of rain gauge network
WMO recommendations

Type of regions	Minimum area for one station under ideal condition in sq.km.	Area to be covered under difficult condition per station in sq.km.
1. Flat regions of temperate and Mediterranean and tropical zones	600-900	900-3000
2. Mountainous regions of temperate and Mediterranean and tropical zones	100-250	250-1000
3. Small mountainous regions with irregular precipitation	25	
4. Arid and polar zones	1500-10000	

Optimum number of raingauge stations

Records from all the existing gauges of a basin help to fix the optimum number of stations. The following statistical analysis helps to obtain optimum number of gauges for a basin on the basis of an assigned percentage of error in estimating the mean areal rainfall.

$$N = \left(\frac{C_v}{E_p}\right)^2$$

N = optimal number of stations
E_p = allowable percentage of error in estimating the mean areal rainfall C_v = Coefficient of variation of the rainfall from existing stations Method to calculate coefficient of variation
Mean of rainfall: $P_{av} = \frac{1}{n}\sum P_i$
Standard deviation: $\sigma = \sqrt{\frac{1}{n-1}\sum(P_i - P_{av})^2}$

Coefficient of variation: $C_v = \frac{\sigma}{P_{av}} \times 100$

Ep is generally taken as 10%.

If Cv is less than 10%, the existing number of stations is assumed to be sufficient. For N>n, the additional stations required for the basin are N-n. Annual rainfall data is normally used in this analysis.

Normal rainfall

Average rainfall for 30 year period

Estimation of missing precipitation

Two commonly used methods

Arithmetic average method

This method is used if the normal annual rainfall of missing station is within 10% of the normal annual rainfall of surrounding stations, data of at least 3 surrounding stations (index stations) are available and the index stations should be evenly spaced around missing station and should be as close as possible.

The formula for computing rainfall of missing station is

$$P_x = \frac{1}{n}(P_1 + P_2 + \cdots + P_n)$$

$P_1, P_2, \ldots, P_n$: rainfall of index stations P_x: rainfall of missing station

n: number of index stations

Normal ratio method

This method is used if the normal annual rainfall of index stations differs by more than 10% of the missing station. The rainfall of surrounding index stations is weighed by the ratio of normal annual rainfall by using the following equation:

$$P_x = \frac{1}{n}\left(\frac{N_x}{N_1}P_1 + \frac{N_x}{N_2}P_2 + \cdots + \frac{N_x}{N_n}P_n\right) = \frac{N_x}{n}\left(\frac{P_1}{N_1} + \frac{P_2}{N_2} + \cdots + \frac{P_n}{N_n}\right) =$$

$P_1, P_2, \ldots, P_n$: rainfall of index stations P_x: rainfall of missing station

n: number of index stations

N_x: normal annual rainfall of missing station

$N_1, N_2, \ldots, N_n$: normal annual rainfall of index stations

Double mass curve analysis for correction for data inconsistencies

The plot of accumulated annual rainfall of a particular station versus the accumulated annual values of mean rainfall of surrounding stations is called double mass curve. This technique is used to check the consistency of rainfall data and to correct erroneous rainfall data. This technique is based on the principle that a group of sample data drawn from its population will be the same.

The reasons for inconsistency are:

Shifting of gauge

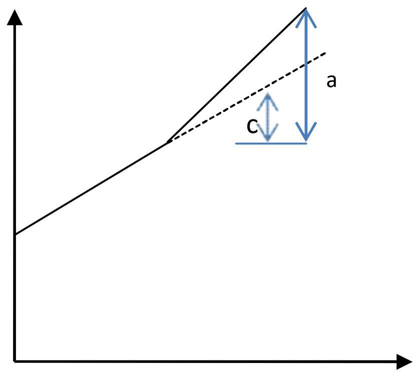
Change in site conditions due to calamities, e.g. fires, landslide

Change in observational procedure

Observation error

If the double mass curve is straight line, the rainfall of the particular station is said to be consistent. If there is break in the slope of the plot, then the rainfall of that particular station is inconsistent. Starting year of change of regime of rainfall is marked by the starting point of the break in slope. Correction has tobe applied beyond the period of change of regime.

Cumulative rainfall of base station



Cumulative of mean rainfall of neighbouring stationsFig. 2.5: Double mass curve
Steps in double mass construction:
Select a group of 5 to 10 neighbouring stations
Arrange data in chronological order with latest data in the beginning.
Compute cumulative rainfall of base station (station whose consistency is to be checked).
Compute cumulative of mean rainfall of neighbouring stations.
Plot cumulative rainfall of base station versus cumulative of mean rainfall of neighbouring stations, and join the points by straight line.
Check if there is break in straight line.

Formula for correction for rainfall after break in line is given by

$$P_{cx} = P_x \frac{M_c}{M_a}$$

P_{cx} = Corrected precipitation for station xP_x:
Original precipitation of station x
M_c: Slope of original line
M_a: Slope of line after change of regime
If c and a are vertical intercept of original line and line after change

$$P_{cx} = P_x \frac{c}{a}$$

A change in slope is normally taken as significant only where it persists for more than five years.Correction should be applied for change in slope exceeding 10% of original line.

Presentation of rainfall data
Rainfall depth
Rainfall intensity: rainfall depth/time interval

Point rainfall
Rainfall data of a station of certain duration
Duration: Hourly, Daily, Weekly, Monthly, Seasonal, annual
Plot: Rainfall versus time in bar diagram

Moving average
Average of consecutive interval
Interval: 3-5 year
Purpose: to isolate the trend in rainfall data and to smoothen out the high frequency fluctuations

Mass curve
Plot of accumulated rainfall versus time
Useful to identify intensity, duration, magnitude, starting and ending time of rainfall
Magnitude = cumulative rainfall at t- cumulative rainfall at t-1
Intensity = slope of curve
Cumulativerainfall

Mass curve

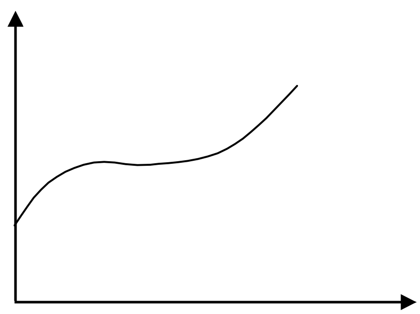


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Fig. 3.6: Mass curve of rainfall

Hyetograph
Plot of rainfall intensity or rainfall depth versus time interval in the form of bar graph
In each bar, time interval between two points is shown in X-axis and corresponding rainfallrepresents Y-axis. From mass curve, rainfall of certain interval dt can be computed and rainfall intensity can beobtained.
The graph represents the characteristics of storms and useful in predicting floods.
Area under hyetograph: total rainfall

Rainfallintensity

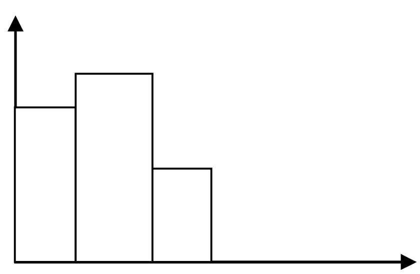

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Fig. 3.7: Hyetograph

Method of computing average rainfall
Three common methods
Arithmetic mean method

This is the simplest method for computing mean rainfall. This method is satisfactory if the gauges are uniformly distributed and the individual gauge catches do not vary greatly about the mean. The formula for computing mean rainfall is

$$P_{av} = \frac{1}{n}\sum_{i=1}^{n} P_i$$

P_{av} = average rainfall
n = number of stations
P_i = precipitation of station i

This method gives equal weights to each gauge. It gives only rough estimate. It does not take into account the topography and other influences. For this method, only the gauges inside the basin are considered.

Thiessen polygon method

In this method, weightage is given to all the gauges on the basis of their areal coverage. Thiessen method assumes that rainfall at any point within the polygon is same as that of the nearest gauge. For this method, all the gauges in and around the basin are considered.

Method of constructing Thiessen polygon
Draw map of basin and locate the rainfall stations.
Connect the adjacent rainfall stations by straight lines, forming triangles.
Draw perpendicular bisectors to each of the sides of triangles lines. The perpendicular bisectors forms boundary of polygons. Wherever the basin boundary cuts the bisectors, it is taken as the outer limit of the polygon.
Measure area of each polygon which surrounds a station. Area can be computed by using formulae for regular geometric figure. For irregular figure, area is determined by using planimeter or by tracing on graph and counting squares.

Thiessen polygon

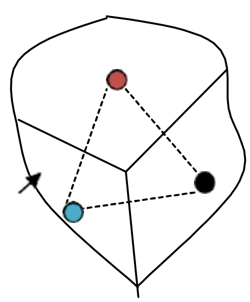


Fig. 3.8: Example of Thiessen polygon

The formula to compute mean rainfall is

$$P_{av} = \frac{\sum_{i=1}^{n} A_i P_i}{A}$$

P_{av} = average rainfall
P_i = Rainfall of station i
A_i = Area of polygon which encloses station i

A= Total basin area
A_i/A: Weight for station i = w_i

$$P_{av} = \sum_{i=1}^{n} w_i P_i$$

Advantages
Use of data nearby stations located outside basin
Consideration of spacing of stations
Easy to perform computation through computer software

Limitations
It does not consider orographic and topographic effects.
The method assumes linear variation of precipitation between stations.

Isohyetal method

An isohyet is a line joining points of equal rainfall. For this method, rainfall stations lying within basin as well as nearby stations around the basin are considered.

Method of constructing isohyetes
Draw map of basin and locate the rainfall stations.
Mark the depth of rainfall at each station.
Draw isohyetes by interpolating between adjacent gauges and considering orographic, stormcharacteristics and other factors.
Measure the area between successive isohyets.

The mean rainfall is computed by

$$P_{av} = \sum \frac{A_j \frac{P_i + P_{i+1}}{2}}{A}$$

P_{av} = average rainfall
Pi, Pi+1: rainfall of isohyets i and i+1
A_j = Area enclosed by isohyets i and i+1A=
Total basin area
Note: if the isohyets go out of the boundary, then the catchment boundary is used as the bounding line.

IsohytesGauges

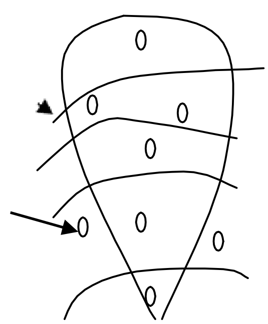


Fig. 3.9: Example of isohyetal method

Advantages
Data from nearby stations located outside the basin can also be used.
Spacing of station as well as magnitude of precipitation is considered in the method.
The method is more accurate due to the consideration of topography and other influences.

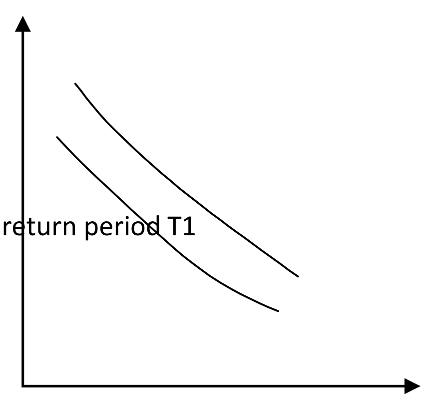
Limitations
The method requires dense gauge network.
Isohyets need to be drawn for each storm.

Intensity duration frequency (IDF) curve
An intensity duration frequency (IDF) curve is a three parameter curve in which duration is taken on x- axis, intensity on y-axis and the return period or frequency as the third parameter. The IDF curve is a very important tool for determination of runoff, which is required for design. The curve can be used to determine the rainfall intensity for other durations with given intensity of a particular duration.

IDF Curves by frequency analysis
When observed rainfall data of different durations are available, IDF curves can be developed using frequency analysis.
Method
For each duration selected e.g. 15 min, 30 min, 1hr etc., extract annual maximum rainfall data from historical record.
For each duration, perform frequency analysis i.e. find rainfall value of different return periods such as 2, 5, 10, 25, 50, 100 yr. (Return period is the average interval of time within which an event of given magnitude will be equaled or exceeded.)

The steps involved in simple plotting position method, which can be used for precipitation data, are given below.
Prepare data of maximum intensity for different durations for different years.
Arrange data in descending order.
Assign rank of data. Assign 1 for highest data, 2 for second highest data and so on.
Calculate the return period of each data. According to California formula, return period (T) = n/m where n = number of data, m = rank.
Plot rainfall versus return period and extrapolate to get rainfall of higher return periods.

Rainfall intensity



curve for return period T2 curve for return period T1

Duration

Fig. 2.10: IDF curve

Intensity of rainfall decreases with the increase in duration of storm and increases with the increase infrequency of storm. IDF curves can be expressed as equations in the exponential form given by

$$i = \frac{KT^x}{(D+a)^n}$$

i= intensity

T = return period or frequencyD = Duration

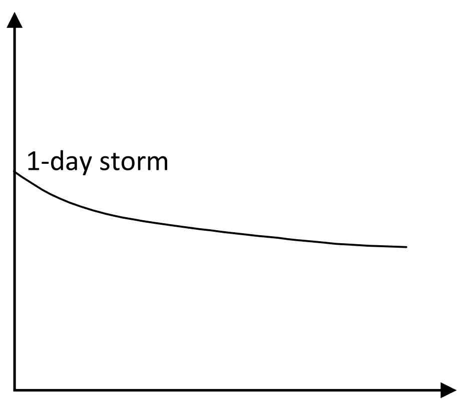
K, x, a, n = Constants

Depth Area Duration (DAD) Curve

A Depth-Area-Duration (DAD) curve is a graphical representation of depth of precipitation and area of itscoverage with duration of occurrence of storm as third parameter. Storms of smaller duration has smaller depth and the depth decreases with increase in area.

The purpose of DAD curve is to determine the maximum amount of precipitation that have occurred over various sizes of drainage area during the passage of storm periods of say 6hr, 12hr, 24hr or other durations. Such information is essential for the design of hydraulic structures such as reservoir, dam, culverts etc.

Depth of precipitation



Area

Fig. 2.11: DAD curve

Incremental isohyetal method is generally used to construct DAD curve. The procedure of this method isas follows.

Identify all the major storms.

Note down the duration for all storms, such as 1 day duration.Prepare isohyetal pattern for all 1 day storms on map.

Take one 1 day storm and calculate the area bounded within the highest isohyets by planimetering. Next compute the area bounded between the largest and second largest isohyets by planimetering. Repeat the procedure for remaining isohyets. Compute average isohyetal depth (P_{mi}). This procedure is repeated for all other 1 day storm of the area. The average depth of precipitation is computed as follows.

$$d_n = \frac{\sum_{i=1}^{n} P_{mi} A_i}{\sum_{i=1}^{n} A_i}$$

where d_n = average depth of precipitation covering up to nth isohyets, P_{mi} = average isohyetal depth for i^{th} isohyets, A_i = Area enclosed between i^{th} and $(i+1)^{th}$ isoheyts.
Plot a graph between cumulative area as abscissa and maximum average depth of precipitation as ordinatecovering the depth area data of all 1 day storms.
Same procedure as explained above is applied for other durations.

DAD curve can be expressed by empirical equation as
$P = P_0\ e^{-KAn}$
P= depth of rainfall over an area of A, P_0 = rainfall at center of rainfall, k and n = constants

Snowfall and its measurement
The atmospheric requirements for snow fall
Presence of water vapor
Presence of ice nuclei
Ambient temperature below 0^0C

Ice nuclei: particles that cause ice crystals to form through either direct freezing of cloud droplets or freezing of water deposited on the particle surface as vapor. E.g., dust particles, combustion products, organic matter

Once ice crystals form, they may splinter and create large number of nuclei to aid the precipitationprocess. Continued growth of an ice crystal leads to the formation of a snow crystal.
Snow pellets
Snowflake: aggregation of snow crystals, may grow in size during its falling

Snowfall or rainfall from snowflake depends upon extent and temperature of layers of air through which itfalls.

Variables
Depth of snow
Snow water equivalent
Snow water equivalent is the amount of water that would be obtained if the snow were melted.
Density of snow
It is the percentage of snow volume that would be occupied by its water equivalent.Density = volume of melt water from a snow sample/intial volume of sample

Snow water equivalent = Depth of snow x Density of snowDensity: in terms of fraction
If density of water and density of snow is given,
Snow water equivalent = Depth of snow x (Density of snow/Density of water)

Density of new snow: 0.01-0.15, damp new snow: 0.1-0.2, settled snow: 0.2-0.3Measurement of snow

Point snowfall water equivalent: Snow is collected in raingauge (non-recording, weighing type), whichis melted and equivalent amount of water is measured.
Point snowfall depth: Ruler, snowboard
Snow accumulated on the snow board is measured by scale or ruler
Snow depth for area having large accumulation: permanent snow stakes (calibrated wooden posts fixed on the ground)
Areal snow cover depth and density: Snow survey
Snow survey means surveying of sections of snow cover to determine depth and density. Depth determination: by preinstalled gauges
Density determination: boring a hole through the snowpack or into the pack and measuring the amount of liquid water obtained from the sample.

Hydrological Losses

Different losses

The difference between precipitation and runoff can be treated as hydrological losses.

Initial losses (interception and depression storage)Interception

Interception is that part of precipitation which is caught and held by the vegetation or obstruction. Much of the intercepted water returns to the atmosphere by evaporation. The remaining part may drip off or flow down through the stem to reach the ground surface. About 10 to 20% of total rainfall is considered as interception losses. Its exact estimation is difficult.

Depression storage

After precipitation of a storm reaches the ground, some part of it is stored in the depressions on the ground surface, which is called depression storage. The amount is eventually lost to runoff through process of infiltration and evaporation and thus forms a part of the initial loss.

The depression storage depends upon

The type of soil

The condition of the surface reflecting the amount and nature of the depression.

The slope of the catchment

The antecedent precipitation, as a measure of soil moisture.

Evaporation

The process by which liquid is converted to vapor is called evaporation. Evaporation occurs from water bodies as well as from soil moisture.

Transpiration

The emission of water vapour from plant leaves is called transpiration.

Infiltration

Infiltration is the process by which water from the ground surface enters into the soil. Infiltration is responsible for recharging groundwater and for maintaining soil moisture.

Evaporation process

Meteorological parameters

Temperature

Temperature is a measure of hotness of an object. The temperature of a locality is a complex function of several variables such as latitude, altitude, ocean currents, distance from sea, winds, cloud cover, and aspect (land slope and its orientation).

Lapse rate

The rate at which temperature decreases with increase in altitude is called lapse rate. It is about 6˚C per 1000 m within the troposphere.

Terminologies for expressing temperature
Mean daily temperature: Average of hourly temperature, if hourly data are available
Maximum daily and minimum daily
Average of the daily max and min temperature, if only maximum and minimum data are available
Normal temperature: Arithmetic mean temperature based on previous 30 years' data
Normal daily temperature: The average mean daily temperature of a given date computed for a specific 30-year period.
Mean monthly temperature: average of the mean monthly maximum and minimum temperature.
Mean annual temperature: average of the monthly means for the year.

Temperature measurement
Using thermometer
The maximum-minimum thermometers for daily maximum and minimum temperature.

Humidity

Amount of water vapor in air is called humidity. Humidity is closely related to its temperature- higher the air temperature, more vapor the air can hold. For this reason, saturation vapor pressure goes up with air temperature.
Saturation vapor pressure
Pressure at which air is saturated with water is called saturation vapor pressure. It is a function of temperature.

$$e_s = 611\, exp\left(\frac{17.27T}{237.3 + T}\right)$$

e_s = saturation vapor pressure (N/m^2)
T= Temperature (^0C)

Significance of Humidity: The amount of water vapor in air effectively controls the weather condition by controlling evaporation from land and water surfaces.

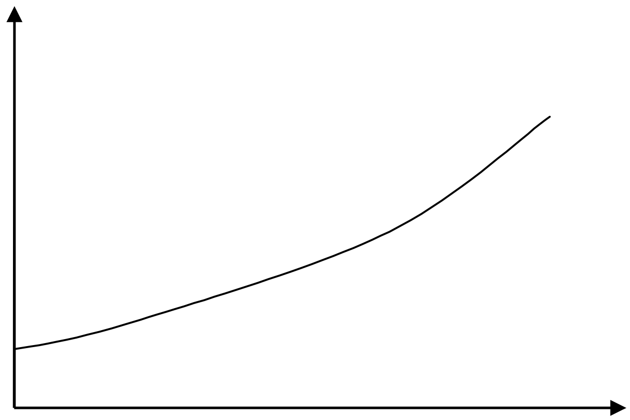
Commonly used measures of humidity
Vapor pressure: partial pressure exerted by water vapor vapor in air on the earth's surface due to its own weight
Absolute humidity: mass of water vapor contained in a unit volume of air at any instant
Specific humidity: mass of water vapor per unit mass of moist air
Relative humidity: Actual vapor pressure (e_a)/ Saturation vapor pressure(e_s)
(or ratio of the amount of water vapor actually contained per unit volume to the amount of water vapor that it can hold at the same temperature when saturated)
Mixing ratio: mass of water vapor per unit mass of perfectly dry air in a humid mixture

Saturation vapor pressure gradient
Gradient/slope of saturation vapor pressure (e_s) curve is found by differentiating e_s with respect to temperature.

$$\Delta = \frac{de_s}{dT} = \frac{4098e_s}{(237.3 + T)^2}$$

Δ= slope of saturation vapor pressure, T = temperature in ^{0}C

Saturation vapor pressure



TemperatureFig. 3.1: Saturation vapor pressure curve

Dew point temperature

The temperature at which air becomes saturated when cooled at constant pressure and moisture content is called dew point temperature.

Measurement of humidity

By using psychrometer: It contains wet bulb thermometer (continuous moisture supply by wrapping with wick and submerging the other end in distilled water) and dry bulb thermometer (recording ambient air temperature).

By using hygrograph: Automatic recording of humidity

Principle: hair reacts to the changes in air humidity by expanding or contracting

Wind

Wind is a moving air. Wind has both speed and direction. Wind direction is the direction from which it is blowing. Wind speed varies with height above the ground. Wind is one of the major factors that affect the climate and evaporation rate from water surface. Wind influences the ability to transport vapor away fromthe surface as well as the temperature of the area. Higher wind speed results in higher evaporation rate from a water surface as the wind replaces saturated air just above the water surface by unsaturated air.

Wind speed is measured by anemometers. For comparable data, all anemometers are installed at same elevation above ground. Wind speed varies greatly with height above the ground due to ground friction, trees, buildings and other obstacles.

Wind speed at a certain height is computed by power law as

$$\frac{V}{V_0} = \left(\frac{Z}{Z_0}\right)^{0.15}$$

V = velocity at any height Z
V_0 = Observed velocity at height Z_0

Types of wind

Sea and land breezes: See breeze is the blowing of wind from sea to land due to higher temperature (lower atmospheric pressure) at land during day time. Sea breeze is the reason we feel cooler near large water body at day time in a hot day. Land breeze is the blowing of wind from land to sea due to quicker cooling of land, and hence denser air above land surface.
Monsoon (seasonal) Winds: Winds whose direction depends on season.
Cyclone (hurricane/typhoon): Cyclones are caused when a low pressure area is surrounded by high pressure areas around which air flows anticlockwise in the northern hemisphere and clockwise in southernhemisphere. A cyclone is generally followed by heavy rain.
Anticyclone: Anticyclone is a region of high pressure surrounded by low areas around which air flows clockwise in the northern hemisphere and anticlockwise in southern hemisphere.
Tornadoes: Tornadoes are similar to cyclone, but they generally form over ocean. Tornadoes are generally destructive to land and property.
Local winds: They affect only limited areas and blow for short durations. The cause of local winds is mostly local temperature depressions.

Radiation

Radiation is the direct transfer of energy by means of electromagnetic waves. Radiation from the sun is called solar radiation. Solar Radiation provides the fuel for the hydrologic cycle. Solar radiation determines weather and climate of earth.

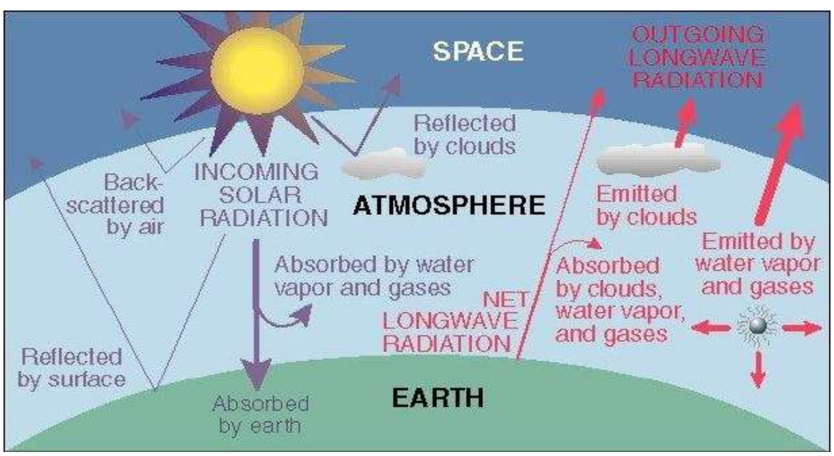


Fig. 3.2: Components of radiation balanceTerminology
Insolation: incident solar radiation

Short wave and long wave radiation
Solar radiation from the sun is referred to as short wave radiation. The radiation from the earth is referredto as long wave radiation.

Albedo
When radiation strikes a surface, it is either reflected or absorbed. The ratio of amount of solar radiationreflected by a body to incoming radiation is called albedo.

Net radiation
The net radiation is the difference between absorbed radiation and emitted radiation.

$r = \frac{R_r}{R_i}$

$R_r = rR_i$

r = albedo

R_r= Reflected radiationR_i =Incoming radiation

Absorbed radiation=$R_i - rR_i = R_i(1 - r)$

Emitted radiation =R_e

Net radiation = $R_i(1 - r) - R_e$

Radiation emission is governed by Stefan-Boltzmann law:

Radiation is also continuously emitted from all bodies at rates depending on the temperatures.

$R_e = e\sigma T^4$

E = emissivity of the surface

σ= Stefan-Boltzmann constant

T = Absolute temperature of the surface (K)

Net radiation at the Earth's surface
Incoming radiation = Long wave radiation (R_l) + Shortwave radiation (R_s)Radiation emitted by the earth = R_e

Albedo = r

Net radiation at the Earth's surface (R_n) is given by

$R_n = (R_l + R_s)(1 - r) - R_e$

Intensity of solar radiation depends on
Scattering in the atmosphere
Absorption by clouds
Obliqueness of the Earth's surface to the incoming radiation (a function of latitude, season andtime of day)

Radiation measurement

Actinometers and radiometers are used to measure intensity of radiant energy. The data is used in studies of evaporation and snowmelt.

Factors affecting evaporation

Meteorological factors

Radiation: most important factor as it directly influences the temperature of the evaporating surface.

Temperature: Increase in temperature increases the evaporation rate but not always proportionally. For same temperature, evaporation in colder months is less than summer months due to other environmental factors.

Humidity: Humidity influences vapor pressure deficit which governs the rate of evaporation.

Vapor pressure: Evaporation is proportional to the difference between saturation vapor pressure at the water temperature and actual vapor pressure in the air.

Wind: Wind helps to carry away moisture as it evaporates and thus accelerates the rate of evaporation. Generally the rate of evaporation increases with the wind speed up to a critical speed beyond which any further increase in wind speed has no influence on the rate of evaporation. There is a relation between wind speed and size of water surface.

Atmosphere pressure: Increase in atmosphere pressure decreases the rate of evaporation.

Nature of evaporating surface

Soil: the rate of evaporation from soil depends on the availability of water, e.g, higher rate for wetsoil, lower rate for dry soil.

Snow and ice: Evaporation from snow can occur when the vapor pressure of the air is less than that of the snow surface i.e. only when the dew point is lower than the temperature of the snow.

Reservoir: The rate of evaporation from a reservoir depends on the heat storage capacity, e.g. for deep water bodies, large heat storage during summer causing less evaporation and vice versa in winter.

Quality of water: Soluble salts reduce the vapor pressure, and thus reduce the rate of evaporation.

Methods of estimation of evaporation

Empirical equations

Empirical equations used for estimating evaporation are functions of saturation vapor pressure at the water temperature (e_s) and actual vapor pressure in the air (e_a).

General equation (Dalton's law): $E = kf(u)(e_s - e_a)$

E= evaporation k = coefficient

f(u) = wind speed correction function e_s = saturation vapor pressure

e_a = actual vapor pressure

Meyer's formula

$$E = C\left(1 + \frac{U}{16}\right)(e_s - e_a)$$

E= Evaporation (mm/day)

U= monthly mean wind speed in km/h measured at 9m above ground C = coefficient (0.36 for large lakes, 0.50 for shallow lakes)

e_s = saturation vapor pressure (mm of Hg) e_a = actual vapor pressure (mm of Hg)

Rhower's formula

$E = 0.771\ (1.465-0.000732P)\ (0.44+0.0733U)\ (e_s-e_a)$

E= Evaporation (mm/day)

P= mean barometric reading in mmHg

U = mean wind velocity at 0.6m above ground in km/he_s = saturation vapor pressure (mm of Hg)

e_a = actual vapor pressure (mm of Hg)

Analytical methods

Water budget method

∑Inflow -∑Outflow =Change in storage + Evaporation loss

$E = \sum I - \sum O \pm \Delta S$

General equations

$E= (P+I_{sf}+I_{gf})-(O_{sf}+O_{gf}+T)\pm\Delta S$

P= precipitation

I_{sf} = Surface inflow

I_{gf} = Groundwater inflow O_{sf} = Surface water outflowO_{gf} = Groundwater outflowT = Transpiration loss

ΔS = Change in storage

Measurement of I_{gf}, O_{gf} and T is not possible, these can only be estimated.

T is usually negligible.

Water budget equation gives approximate values.

Energy budget method

Based on law of conservation of energy

Incoming energy = outgoing energy + Change in stored energy

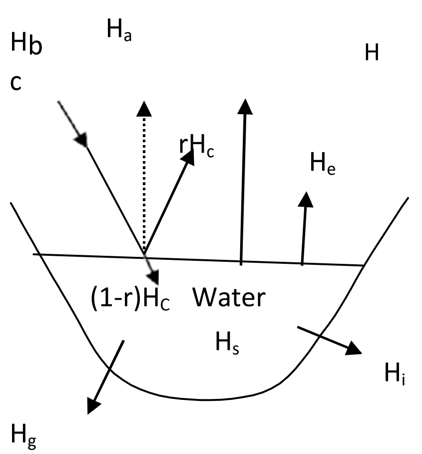


Fig. 3.3: Components of energy balance

Energy balance to evaporating surface in a period of one day $H_n = H_a + H_e + H_g + H_s + H_i$
H_n = net radiation = $H_c(1-r) - H_b r$ H_c = Reflected radiation
H_c= Incoming solar radiation R =albedo
H_b = Back (Long wave) radiation from water body H_a = Sensible heat transfer from water surface to air H_e = Heat energy used up in evaporation
H_g = Heat flux into the ground H_s = Heat stored in water body
H_i = Net heat conducted out of the system by water flow (advected energy)

For short time period H_s and H_i can be neglected. All the terms except H_i can either be measured orevaluated indirectly. H_a is estimated using Bowen's ratio.

The ratio of sensible heat flux to heat flux used up in evaporation is called Bowen ratio.

$$\beta = \frac{H_a}{H_e} = \frac{H_a}{\rho . E . L}$$

β = Bowen ratio
ρ = Density of water E = Evaporation
L = Latent heat of vaporization
Estimate of β

$$\beta = \gamma \frac{Tw - Ta}{ew - ea}$$

γ = Pshcrometric constant
ew = saturated vapor pressure (mmHg)
ea = actual vapor pressure (mmHg) Tw = Temperature of water surface(C) Ta = Temperature of air (C)

$$E = \frac{H_n - H_g - H_s - H_i}{\rho L(1 + \beta)}$$

Mass transfer method

This method is based on theories of turbulent mass transfer in boundary layer to calculate the mass water vapor transfer from the surface to the surrounding atmosphere.

Evaporimeters (Evaporation pan)

Evaporation Pan, also called Evaporimeter, is shallow vessels containing water. These are placed in open to measure the loss of water by evaporation. Water is placed in the evaporation pan and the change in depth of water due to evaporation is measured.
Lake or reservoir evaporation = Pan coefficient x Pan evaporation

Pan Evaporation differs from lake evaporation due to the depth of exposure of pan above ground, color of the pan, height of the rim, heat storage and heat transfer capacity with respect to reservoir, variation in vapor pressure, wind speed and water temperature. Pan coefficient takes into account these factors.
Pan coefficient: 0.6 to 0.8

Various types of pans Class A evaporation pan
It consists of a cylindrical vessel made of galvanized iron sheet. The pan is placed 15cm above the groundsurface in such a way that it gets free circulation of air.

Sunken pan (Colorado Sunken pan)
The pan is buried into the ground such that the water level is at the ground level. Advantage of this pan is that the aerodynamic and radiation characteristics are closer to the reservoir. The water level is maintained at or slightly below the ground level.

Evapotranspiration
The processes of evaporation from the land surface and the transpiration from the vegetation arecollectively termed evapotranspiration (ET).
Main factors affecting ET
Supply of energy (solar radiation)
Ability to transport vapor away (wind speed and humidity gradient)
Supply of moisture at the evaporating surface Potential Evapotranspiration and Actual Evapotranspiration
Potential Evapotranspiration (PET) is the evapotranspiration that would occur from a well vegetated surface when moisture supply is not limiting. The real evapotranspiration occurring in a specific situation is called actual evapotranspiration (AET).

Field capacity and permanent wilting point

Field capacity is the maximum quantity of water that the soil can retain against the force of gravity. Permanent wilting point is the moisture content of a soil at which the moisture is no longer available in sufficient quantity to sustain the plants. The difference in these two moisture contents is called available water.

If the water supply to the plant is adequate, soil moisture will be at filed capacity and AET = PET.If the water supply is less than PET, the soil dries out and AET<PET.
At permanent wilting point, AET = 0

Penman method for determination of evapotranspiration

Penman method is a combined aerodynamic and energy balance method for estimating evapotranspiration. Evapotranspiration is computed by aerodynamic method when energy supply is not limited and by the energy balance method when vapor transport is not limited. But, normally, both of these factors are limiting, so a combination of the two methods is needed.

Assumptions:
Steady state energy flow prevails.
Changes in heat storage over time in the water body are not significant.
Vapor transport coefficient is a function of wind speed.
Advected energy input is small, which may be neglected.

Penman's formula for estimation of evapotranspiration is given by

$$PET = \frac{AH_n + \gamma E_a}{A + \gamma}$$

PET = daily potential evapotranspiration (mm/day)A = slope of saturation vapor pressure (mmHg/^{0}C)H_n =Net radiation (mm/day)
E_a = Evaporation due to aerodynamic method (mm/day)
γ = Psychrometric constant (mmHg/^{0}C) (can be taken as 0.49mmHg/^{0}C)

The net radiation is estimated by the following equation:

$$H_n = H_a (1 - r) \left(a + b \frac{n}{N}\right) - \sigma T_a^4 (0.56 - 0.092\sqrt{e_a}) \left(0.10 + 0.90 \frac{n}{N}\right)$$

H_a = Incident solar radiation outside the atmosphere on a horizontal surface (mm/day)a = constant depending upon latitude ϕ and is given by a = 0.29Cos ϕ
b = constant with an average value of 0.52
n = actual duration of bright sunshine hours (hours)
N = Maximum possible hours of bright sunshine (hours) (function of latitude)r = albedo
σ = Stefan-Boltzman constant = 2.01x10^{-9} mm/day T_a = mean air temperature (degree Kelvin) = 273+^{0}C e_a = Actual vapor pressure (mmHg)

E_a is estimated as

$$E_a = 0.35 \left(1 + \frac{u2}{160}\right) (e_s - e_a)$$

u2 = mean wind speed at 2m above ground (km/day)
e_s = Saturated vapor pressure at mean air temperature (mmHg)e_a = Actual vapor pressure (mmHg)
For the computation of PET, data on temperature, wind speed, radiation (or sunshine hours) and vaporpressure (or humidity) are needed. H_a, N and A are obtained from tabulated values, or from equations.

Value of e_s from T
$$e_s = 4.584\, exp \left(\frac{17.27T}{237.3+T}\right)$$
e_s = saturation vapor pressure (mmHg)
T= Temperature (^{0}C)
If Relative humidity (RH) is given, RH = e_a/e_sEquation to compute A
$$A = \frac{4098 e_s}{(237.3+T)^2}$$ where e_s = saturation vapor pressure (mmHg), T= Temperature (^{0}C)
For 20^{0}C, A = 1.08 mmHg/^{0}C

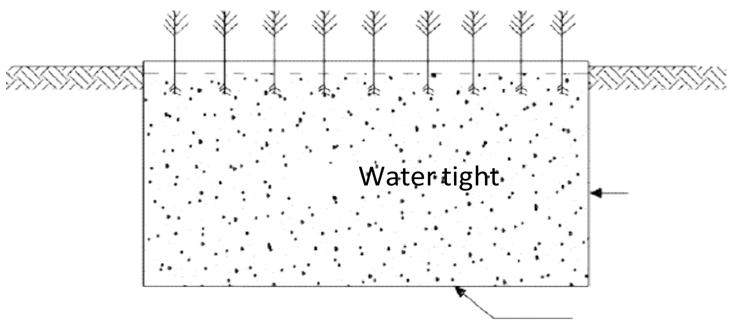
Value of r: Water surface = 0.05, Bare land: 0.05-0.45

Measurement of evapotranspiration

Lysimeter Method

Lysimeter is a small tank containing soil in which the plants are grown. It is generally cylindrical tank about 60 to 90 cm in diameter and 180 cm deep. This tank is buried in ground such that its top is made like the surrounding ground surface. Water is applied to the lysimeter for the satisfactory growth of plant. Percolated water excess to the plant use is collected in a pit and Evapotranspiration is obtained.

G.L.



SOIL

Pervious bottom

Fig. 3.4: Lysimeter

Computation of evapotranspiration$P+W=O+ET+\Delta S$

P = precipitation

W = Amount of water applied

O = Quantity of water drained outET = Evapotranspiration

ΔS = Change in soil moisture storage

Infiltration
Introduction

Infiltration is the process by which water enters the soil from the ground surface. Infiltration first replenishes the soil moisture deficiency. The excess water then moves downwards by the force of gravity. This downward movement under gravity is called percolation (or seepage). Percolation is thus themovement of water within the soil.

Infiltration rate (f) is the rate at which water enters the soil at the surface. Cumulative infiltration (F) is the accumulated depth of water infiltrated during a given time period.

$$F(t) = \int_{o}^{t} f(t)dt$$

$$f(t) = \frac{dF(t)}{dt}$$

Infiltration capacity (fc) is the maximum rate at which a given soil can absorb water under a given set of conditions at a given time.

The actual rate of infiltration (f) can be expressed asf = fc for i ≥ fc
f = i for i < fc i = intensity of rainfall
Infiltration capacity of a soil is high at the beginning of a storm and has an exponential decay as the timeelapses.

Hydraulic conductivity: It is a measure of ability of the soil to transmit water.
Field capacity: Field capacity is the maximum amount of water that the soil can hold against the force ofgravity.

Moisture zones

Depth

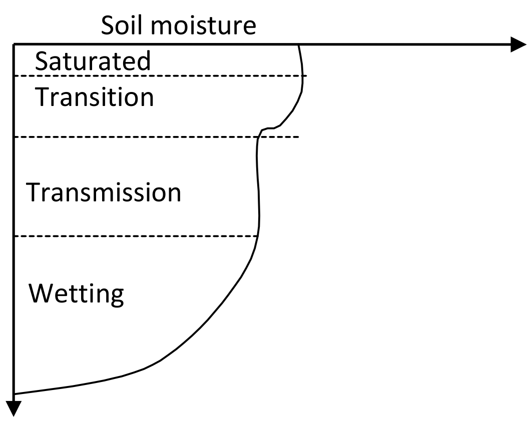


Fig.3.5: Moisture zones
Saturated zone: top zone
Transition zone: second zone
Transmission zone: uniform moisture content, moisture content above field capacity but below saturation, unsaturated zone
Wetting zone: moisture content at or near field capacity, decrease of moisture with depth, wetting front as sharp discontinuity

Factors affecting infiltration (f)
Characteristics of soil
Type of soil, Porosity, texture (determines size of pores), Structure (affects aggregation)
Permeability: high f for loose, permeable sandy soil
Underdrainage: high f for good underdrainage
Grain size of soil particles: higher f for large grain size
layering
Condition of soil surface and its vegetative cover
Low f for bare soil: Clogging the surface by inwashing of fine particles
Grass and vegetation cover: high f
Antecedent moisture content of the soil
Second storm in succession: low f
Climatic conditions
Temperature affecting viscosity and thus f (less viscous, more f)
Rainfall intensity and duration
Intense rainfall: progressive reduction of f due to increased supply of moisture, mechanical compaction and in-wash of finer particles
Sustained heavy rainfall of longer duration: steady reduction in f_c until f attains aconstant value.
Human activities
Crop growing: increase of f
Construction of road, house etc.: reduction of f
Quality of water
Presence of salt: affecting viscosity and reducing porosity, lower f
Turbidity: clogging pore space, lower f
Groundwater table
Close to surface: low f

Horton equation for infiltration
According to Horton, Infiltration begins at some rate f_0 and exponentially decreases until it reaches a constant value f_c
$f(t) = f_c + (f_0 - f_c)e^{-kt}$ f(t): infiltration capacity at any time t from the start of the rainfall f_0: initial infiltration capacity at t = 0
f_c : infiltration rate at the final steady stage when the soil profile becomes fully saturated
k : decay constant depending upon soil characteristics and vegetation cover, known as Horton coefficient Three parameter to fix: f_0, f_c, k, practical difficulty in determination
f

f_0

f_c

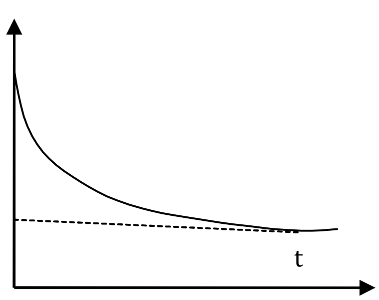


Fig. 3.6 : Infiltration curve

Cumulative infiltration or total infiltration using Horton's equation for time t from start

$$F(t) = \int_{o}^{t} f(t)dt$$

$$= \int_{o}^{t} [f_c + (f_0 - f_c)e^{-kt}]dt$$

$$= f_c t + (f_0 - f_c) \left| \frac{e^{-kt}}{-k} \right|_0^t$$

$$F(t) = f_c t + \frac{f_0 - f_c}{k}(1 - e^{-kt})$$

Average infiltration in time t = F(t)/t = $f_c + \frac{f_0 - f_c}{kt}(1 - e^{-kt})$

Cumulative infiltration or total infiltration depth in between time t1 and t2

$$F(t) = \int_{t1}^{t2} f(t)dt$$

$$= \int_{t1}^{t2} [f_c + (f_0 - f_c)e^{-kt}]dt$$

$$= f_c(t2 - t1) + \frac{f_0 - f_c}{(-k)}(e^{-kt2} - e^{-kt1})$$

To determine k with known values of F(t), f_c, f_o and t

For large t, the value of e^{-kt} becomes negligible. Hence above equation reduces to

$$F(t) = f_c t + \frac{f_0 - f_c}{k}$$

$$k = \frac{f_0 - f_c}{F(t) - f_c t}$$

If rainfall intensity (i) is less than f, all rainfall is infiltrated. Runoff occurs only after i>f.

Determination of constants f_0, f_c and K from given data of f and t

Graphical approach

Plot f on Y-axis and t on x-axis. Draw exponential curve and note down the values of f_0 and f_c.
Horton's infiltration equation is given by $f = f_c + (f_0 - f_c)\, e^{-Kt}$
$f - f_c = (f_0 - f_c)\, e^{-Kt}$ Integrating

$$\int_0^{\infty} (f - f_c)dt = F = Area\ under\ the\ curve$$

$$\int_0^\infty (f_0 - f_c) e^{-kt} \, dt = \frac{f_0 - f_c}{k}$$

Equating above expressions, K can be determined by

$$k = \frac{f_0 - f_c}{F}$$

Statistical approach

Horton's infiltration equation is given by $f = f_c + (f_0 - f_c) e^{-Kt}$

$f - f_c = (f_0 - f_c) e^{-Kt}$

Taking log on both sides $\ln(f - f_c) = \ln(f_0 - f_c) - Kt$

Let $y = \ln(f - f_c)$, $c = \ln(f_0 - f_c)$. Then above equation reduces to $y = -Kt + c$: linear equation

Procedure:

Take f_c from the given data.

Determine K and C by least square method.

$$K = -\frac{N \sum xy - \sum x \sum y}{N \sum x^2 - (\sum x)^2}$$

$$c = \frac{\sum y - (-K) \sum x}{N}$$

With $c = \ln(f_0 - f_c)$, compute f_0.

Excess rainfall or effective rainfall

Infiltration indices

Average rate of infiltration is called infiltration index. For computation of surface runoff and flood discharge, the use of infiltration curve is not convenient. So, we can use constant value of infiltration rate for the duration of storms.

Two common infiltration indices

a. ϕ index

The average rate of rainfall above which the rainfall volume equals to runoff volume is called ϕ index. It is based on the assumption that for a specified storm with given initial conditions, the rate of basin recharge remains constant throughout the storm period. i.e. ϕ remains constant.

For $i < \phi$, $f = i$

For $i > \phi$, runoff = i-fi = rainfall intensity

f = infiltration rate ϕ : total abstractions

The amount of rainfall in excess of the index is known as effective rainfall or rainfall excess.

Method of determination of ϕ index

Given: rainfall hyetograph and direct runoff

Use same unit ,e.g. mm, cm for rainfall and runoff. Take incremental rainfall if cumulative rainfall is given.

Method 1

Trial and error with effective time (t_e)

Consider the whole duration of rainfall as effective in the beginning.

First trial: ϕ= (Total rainfall - Direct runoff)/t_e

where t_e = total time of excess rainfall contributing for direct runoff (effective duration)

Compute rainfall excess of each rainfall pulse and find total rainfall excess.

Rainfall excess = observed rainfall (R) - ϕ Δt where Δt = interval of rainfall data for rainfall intensity> ϕ, 0 otherwise

Compare total rainfall excess with direct runoff. If rainfall excess (Re) is not same as direct runoff (Q), take another value of te. Take te by subtracting ineffective rainfall duration from whole period.

Second trial: ϕ= (Total rainfall - Direct runoff-Ineffective rainfall)/t_e

Repeat steps b-c until Re=Q.

Method 2

Trial and error with ϕ

Consider the whole duration of rainfall as effective in the beginning.

First trial: ϕ= (Total rainfall - Direct runoff)/t_e

where t_e = total time of excess rainfall contributing for direct runoff (effective duration)

Compute rainfall excess of each rainfall pulse and find total rainfall excess.

Rainfall excess = observed rainfall (R) - ϕ Δt where Δt = interval of rainfall data for rainfall intensity> ϕ, 0 otherwise

Compare total rainfall excess with direct runoff. If rainfall excess (Re) is not same as direct runoff (Q), take another value of ϕ.

Re>Q, increase ϕRe<Q, decrease ϕ

Repeat steps b-c until Re=Q.

2. W-index

A w-index is defined as average rate of infiltration during the time rainfall intensity exceeds the infiltration capacity. This index is considered as an improvement over ϕ index in the sense that initial losses (interception and surface storage) are considered.

$$W = \frac{P - R - I_a}{t_e}$$

P = total storm precipitation

R = Total storm runoffI_a = Initial losses

t_e = duration of the excess rainfall (time during which rainfall rate exceeds infiltration rate) W =average rate of infiltration

For I_a = 0 (long and heavy storms) ϕ index = W index

Determination of I_a is difficult. So W_{min} index is used instead of W-index when the soil condition is very wet so that the soil infiltration rate is almost constant and infiltration is at the minimum rate for the basin.

ϕ index and W index depends on soil type, vegetal cover, initial moisture condition, and storm duration and intensity.

Determination of W index

Prepare the rainfall data by deducting the initial loss from the first pulse of rainfall and then follow the same procedure as that of ϕ index.

Measurement of infiltration

Ring infiltrometer

Ring infiltrometer is a metal ring that is driven into the soil. There are two types of infiltrometers:

Single tube infiltrometer: It is a hollow metal cylinder of 60cm long and 30 cm in diameter. Water is placed inside the ring and the level of water is recorded at regular time intervals as it recedes. This data is used to prepare cumulative infiltration curve, from which infiltration capacity as a function of time maybe calculated

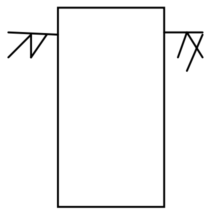


Fig. 3.7: Simple infiltrometer

Double tube infiltrometer: It consists of two concentric hollow cylinders of same length. Water is added to both rings to maintain the same height. The infiltration data from the inner cylinder is taken as infiltration capacity of the soil. The outer cylinder is maintained to prevent spreading of water from the inner one.

Ground

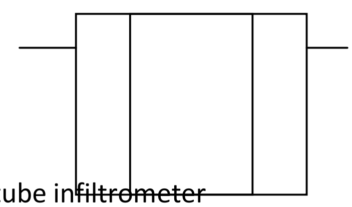


Fig.3.8: Double tube infiltrometer

Rainfall simulator

A rainfall simulator consists of a sprinkler with nozzles capable of producing artificial rain of various intensities, drop sizes and durations. A field plot of about 2mx4m is selected on which the nozzles spray water at a height of 2 m or more to the field. Arrangement is made to collect and measure the runoff from the plot. Experiments are conducted under controlled conditions with various combinations of intensities and durations. Using the water budget equation, infiltration rate is estimated.

$F_d = P_d - S_{rd} - S_{ol}$

F_d = Depth of infiltrated water

P_d = Simulated rainfall depthS_{rd} = Surface runoff depth
S_{ol} = other losses, e.g. depression storage, detention, abstraction

Chapter 4:Surface runoff

Characteristics of drainage basin

Drainage basin/watershed/catchment
Basin area (A): area of land draining into a particular location of a stream

For delineating basin, we need topo map. The map shows changes in elevation by using contour lines. Features of contour
Uphill: contour with higher elevation
Hill: circular contour, ridge: highest point
Saddle: mountain pass
Valley: V or U shaped with the point of the V/U being the upstream end
Close together contours: steep slope
Widely spaced contour: level ground

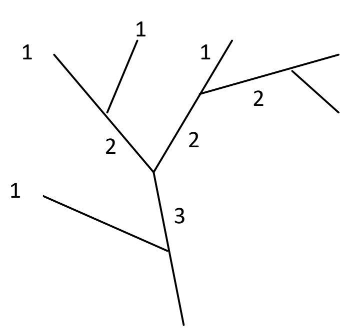
Basin delineation procedure on topo map
Mark the outlet point
Mark the highest point (ridge line: catchment divider) around the river
Start from the outlet and draw line perpendicular to the contours in such a way that the line passes from the highest point (ridge)
Continue to the opposite side of the watercourse, finally ending to the outlet.

Relation of watershed discharge Q with basin area A: $Q = xA^y$

Stream order
measure of amount of branching within a streamStream order assigning procedure
The smallest recognizable channels are designated order 1. (non-branching tributary)
Where two channels of order 1 join, a channel of order 2 results downstream (receiving flow from 1st order). In general, where two channels of order i join, a channel of order i+1 results.
Where a channel of lower order joins a channel of higher order, the channel downstream retainshigher of the two orders.
Order of the basin: order of the stream draining at outlet = highest order in the basin

1

1



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Example of stream order

Variables based on stream ordering

Bifurcation ratio (R_B): ratio of the number N_i, of channels of order i to the number N_{i+1} of channels oforder i+1

$R_B = N_i/N_{i+1}$

R_B: relatively constant from one order to another

Length ratio (R_L): ratio of average length of streams of order i+1 to that of order i$R_L = L_{i+1}/L_i$

Area ratio (R_A): ratio of average area drained by streams of order i+1 to that of order i$R_A = A_{i+1}/A_i$

Drainage density (D_d): ratio of total length of all streams of the basin to its area$D_d = L_s/A$

Indication of drainage efficiency

Higher D_d, quicker runoff, less infiltration and other lossesLength of overland flow = $1/(2\ D_d)$

Length area relationship (Horton's formula): $L = 1.4\ A^{0.6}$ where A- mile^2 (Useful for large riversof the world), L -mile

Stream density (D_s): ratio of number of streams of given order per sq. km.$D_s = N_s/A$

Shape of the basin

Shape of the basin governs the rate at which water enters the stream. The shape of basin is expressed byform factor.

Form factor = average width of basin (B)/axial length of basin (L) = A/L^2 Shape factor (B_s): ratio of square of basin length (L) to its area (A), $B_s = L^2/A$

Slope of the Channel

Slope of channel affects the velocity and flow carrying capacity at any given location at its course. Slope = elevation difference between 2 points of a channel(h)/horizontal length between points (L)

Centroid of basin

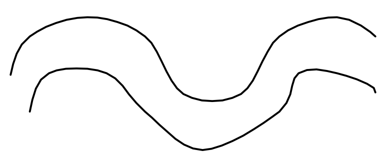
Location of point of weighted center

Hydraulic geometry

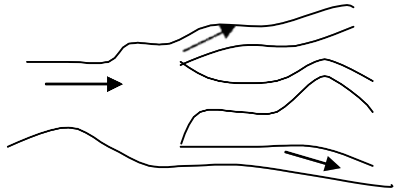
It includes the character of channel, longitudinal variation of mean depth, width and velocity at aparticular cross-section.

Stream pattern

Meandering types - Formation of successive bends of reverse order leading to the formationof a complete S curve called meander.



Braided - Formation of branches separated by islands



Straight - Straight and single channel.

Flood plains

The flood plains of a river are the valley floor adjacent to the channel, which may be inundated duringhigh stage of river. Flood plains are formed due to the deposition of sediment in the river channel and deposition of fine sediments on the flood plains on flooding.

4.10 Factors affecting runoff

Physiographic factors	Climatic factors
Basin characteristics Shape Size Slope Nature of valley Elevation Drainage density Infiltration characteristics land use and cover soil type and geological conditions lakes, swamps and other storage 3. Channel characteristics: cross-section, roughness and storage capacity	Strom characteristics Precipitation: duration, intensity and magnitude Movement of storm Initial loss Evappotranspiration

Shape

Time taken for the water to reach to outlet from remote part depends upon the shape of basin.

Fan shaped: Greater runoff (same size tributaries, almost similar time of concentration)

Elongated: broad and low peak (distributed over time)

Peak flow proportional to square root of drainage areaSize

Small basin: overland flow predominant

Large basins: channel flow predominant, constant minimum flow than small basins

Slope

Slope: control velocity of flow

Related to overland flow, infiltration capacity and time of concentration of rainfall in streams

Large stream slope: quicker depletion of storage

Steeper slope for small basin: higher peak

Elevation

Affects mean runoff (effect of evaporation and precipitation and effect of snow)

Drainage density

Drainage density = total channel length/total drainage area

High density: fast response

Low density: slow response

Land use
Vegetal cover: reduce peak flow
Barren land: high runoff

Soil
Type of soil and subsoil and their permeability conditions
Geology: Controls infiltration

Storage: reduce runoffLakes: reduce flood

Rainfall intensity: increase in runoff with increase in intensityRainfall duration: controls volume of runoff
Rainfall distribution: maximum runoff occurs when entire catchment contributes to runoff.
Direction of the storm movement: affects peak flow and time of duration of runoff, up to down: high runoff, down to up: low runoff
Evapotranspiration: inversely proportional to runoff

Rainfall runoff relationship
Correlation: degree of association between variables.

The relationship between rainfall and runoff is complex due to a number of factors. Therefore, simplemethod like correlating runoff with rainfall is used in practice.

Linear
Equation for straight line regressionR = aP + b
where R = RunoffP = Rainfall
a, b: constants

Coefficients by regression

$$a = \frac{N(\sum PR) - (\sum P)(\sum R)}{N(\sum P^2) - (\sum P)^2}$$

$$b = \frac{\sum R - a \sum P}{N}$$

N = number of observations
Coefficient of correlation

$$R = \frac{N(\sum PR) - (\sum P)(\sum R)}{\sqrt{[N(\sum P^2) - (\sum P)^2][N(\sum R^2) - (\sum R)^2]}}$$

Exponential
For large catchments, exponential relationship can be developed
$R = \beta P^m$
β, m: coefficients
Taking log for linearization

$lnR = mlnP + ln\beta$

With R = lnR, a = m, P = lnP, b = ln β, above equation reduces to the one same as before.

Stream gauging
Streamflow
That part of precipitation which appears in a stream as surface runoff.

Discharge
Volume of water flowing through a channel cross section per unit time.
Hydrometry: science of measurement of waterStage or gauge height
The elevation of water surface at a location in any water body above a reference datum.
Water body: River, lake, canal, reservoir

Stream gauging station
Stream gauging station is defined as the location at which the river discharges are recorded and thedischarge measurements are carried out.
Purpose of stream gauging: to provide systematic records of stage and discharge

Factors to be considered for the selection of site for stream gauging
Easily accessible
Stable and fairly straight river reach about 100m u/s and d/s
Stable and regular channel bed
Free from backwater effects
Regular cross section
No excessive turbulence and eddies
No excessive vegetal or aquatic growth
Velocities: neither too high nor too low, generally in the order of 0.1-5m/s

Stage measurement
Manual or non-recording gauge

Manual gauge is read and recorded by observer/gauge reader once, twice, thrice daily or more. It does notprovide continuous record of stage. It is cheaper and easier to install.

Staff gauge
Staff gauge is the most common and simplest form of manual gauge. It consists of a graduated plate fixed in the stream or on the bank of river or on a structure e.g. bridge abutment or pier. The level of water surface in contact with the gauge is measured by matching the reading of the staff and adding with reference datum level.
It is of three types
Vertical: one vertical gauge
Sectional: more than one gauges at different locations
Inclined

waterlevel

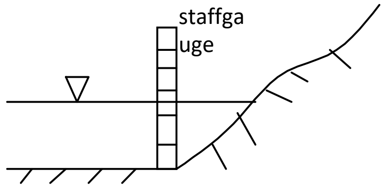


Fig. 4.1: Staff gauge

Recording gauge or automatic gauge

Recording gauge records continuous stage of a river over time.

Two common automatic gauges
Float gauge

A float is connected to one end of a wire which passes through a recorder, and the other end of a rope is balanced by a suitable counterweight. Displacement of float due to rising or lowering of water level causes an angular displacement of pulley and hence of the input shaft of the recorder. Mechanicallinkages convert this angular displacement to the linear displacement of a pen to record over a drum driven by clockwork. The float gauge is protected by installing a stilling well.

Recorder
Weight

Float
Water level

Intakepipes
Intake well

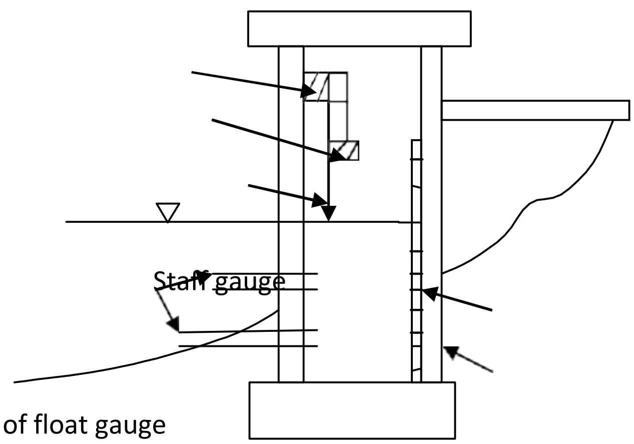
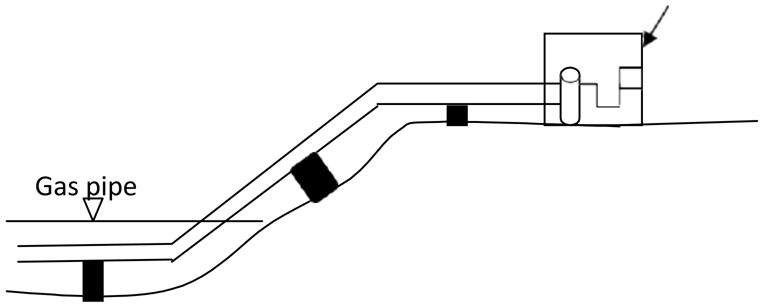


Fig. 4.2: Arrangement of float gauge

Bubble gauge

Bubble gauge consists of small tube placed at the lowest water level through which compressed air (usually CO_2 or N_2 gas) is continuously bubbled out. The pressure required to continuously push the gas stream out beneath the water surface is a measure of depth of water over the nozzle of the bubble stream. This pressure is measured by a manometer in the recorder house.

Recorder house



Water level

River bed

Fig. 4.3: Arrangement of bubble gauge recorder

Discharge measurement using velocity-area method

Velocity area method

This involves the measurement of velocity at the gauging site and the corresponding discharge to obtain river discharge. The velocity is zero at the periphery and changes rapidly as we move from the bank. So a single area-velocity measurement for the entire cross-section will give highly erroneous results. Therefore, the cross-section of a river is divided into a number of subsections by imaginary verticals.

Criteria for selection of verticals

Each vertical should not pass more than 10% of total discharge

Width of each subsection = 5% of total width of river

Difference of velocities in adjacent segments: not more than 20%

Discharge variation between adjacent subsections: between 5% to 10%

For computation of area, the depth of flow is determined by following methods:

Wading or sounding rod: If the river be crossed, a wading rod is used to measure the depth of flow. A man walks across the river section with a graduated wading rod to measure water depth.

Cableway: For deep rivers, cableway is constructed to measure depth and velocity. The lower endof a cable attached to a current meter with a sounding weight is lowered from cablecar. By measuring the length of cable lowered, the depth of flow is measured while velocity is recorded simultaneously by current meter.

Echo-sounder: In this method, high frequency sound wave is sent down by transducer kept immersed at the water surface and the echo reflected by the bed is also picked up by the same transducer. By comparing the time interval between the transmission of the signal and the receipt of its echo, the distance to the bed is obtained. This method is useful for high velocity streams, deep streams and mobile or soft bed streams.

Velocity is measured by current meter or floats.Measurement procedure

Divide the cross-section of the river into n number of verticals.
At each vertical, measure the horizontal distance from the reference bank, the depth of water and the velocity at one or more points.
Compute width, cross-sectional area and average velocity to get discharge at each sub-section. Computation of average velocity in a vertical
One point method: for depth<1.0 m $V_{av} = V_{0.6d}$
Two point method:
$V_{av} = 0.5(V_{0.2d}+V_{0.8d})$
d = depth from water surface

1 2 3 4 5 6 7 8

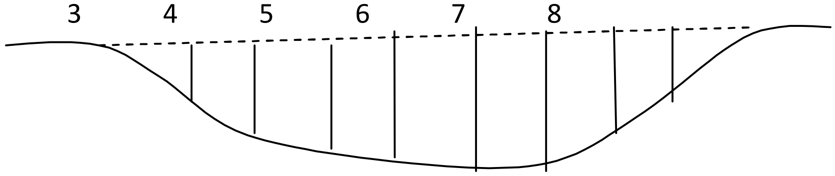


Fig. 4.4: section for area-velocity method

<u>Computation of discharge</u>

Mid section method: widely used
In this method, half width to the left and half width to the right of a vertical is taken as width for a sub-section.
For section 2 to n-1

Width $(\overline{Wav}_i) = \frac{1}{2}(W_i + W_{i+1})$

W_i = Width of section i and W_{i+1} = width of section i+1
For first and last triangular sections

$$Wav_1 = \frac{\left(W_1 + \frac{W_2}{2}\right)^2}{2W_1}$$

$$Wav_n = \frac{\left(W_n + \frac{W_{n-1}}{2}\right)^2}{2W_n}$$

(Alternative way: Width $(\overline{Wav}_i) = \frac{1}{2}(W_i + W_{i+1})$ can be used from section 1 to n.)

Cross-section area (Ai) = $wav_i.d_i$ where d_i = Depth of section i

Discharge at each subsection $(Q_i) = A_i Vav_i$ Total discharge = $\sum Q_i$

Mean section method Cross-section area

$$A_i = \left(\frac{d_i + d_{i+1}}{2}\right) b_i$$

Discharge at each subsection

$$Q_i = A_i \left(\frac{Vav_i + Vav_{i+1}}{2} \right)$$

Total discharge = $\sum Q_i$

<u>Moving boat method for discharge measurement in a deep river</u>

V_b= Velocity of boat at right angle to the streamV_f = Flow velocity
V_r = Resultant velocity
θ = angle made by resultant velocity with the direction of boat
Δt = time of transit between two verticals
Convert surface velocity to average velocity. (V_{av} = 0.85xVsurface)

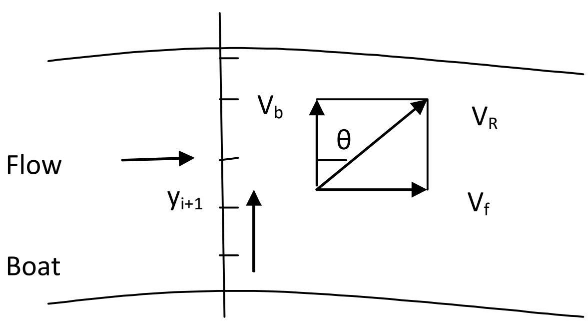


Fig. 4.5: Moving boat method

$V_b = V_R Cos\theta$ and $V_f = V_R Sin\theta$
Discharge in each segment

$$\Delta Q_i = \left(\frac{y_i + y_{i+1}}{2} \right) W_i V_f$$

Where $w_i = V_b \Delta t$
Total discharge = $\sum \Delta Q_i$

Velocity measurement by current meter
Current meter

Current meter is the most commonly used instruments for measuring stream velocity. It consists of a rotating element which rotates due to the reaction of stream current with an angular velocity proportional to the stream velocity. It is weighted down by lead weight called sounding weight to keep in stable position in flowing water.

Types of current meter

Vertical axis meter

It consists of a series of conical cups mounted around a vertical axis. The cups rotate in horizontal plane. The revolutions of cup assembly for a certain time is recorded and converted to stream velocity. The normal range of velocity measured by such current meter is 0.15m/s to 4m/s. This type of current meter cannot be used if the vertical component of the velocity is signigicant.

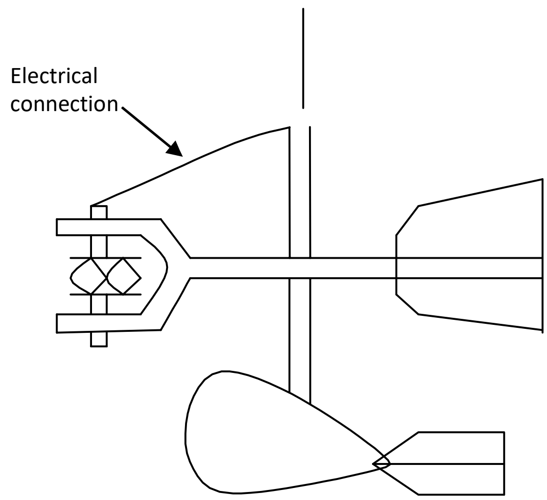


Fig. 4.6: Vertical axis current meter

Horizontal axis meter

It consists of a propeller mounted at the end of horizontal shaft.The revolutions of propeller for a certain time is recorded and converted to stream velocity. The current meter can measure velocity from 0.15m/sto 4m/s. This type of current meter is fairly rugged and is not affected by oblique flows of as much as 15^0.

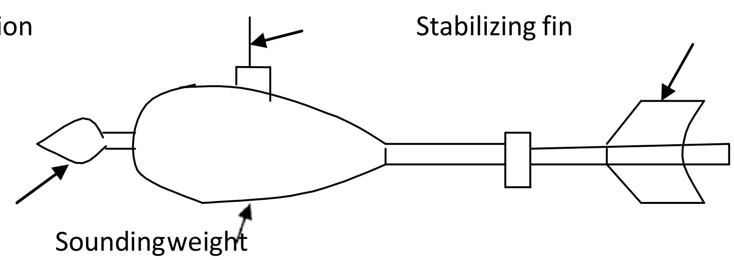


Fig.4.7: Horizontal axis current meter

Relationship between current meter rotation speed and stream velocity

A current meter is so designed that its speed of rotation varies linearly with stream velocity (V). Therelationship is
$V = a\,N_s + b$
where V= stream velocity (m/s)
N_s = revolutions per second of current metera, b = constants

Calibration of current meter

Determination of constants a and b is known ascalibration of current meter.

Current meters are calibrated in ponds or long channels where water is held stationery. A vehicle with cantilever arm projection to the channel helps to lower and move the current meter in the pond water. For each run, the current meter is moved at a predetermined speed (v) and the number of revolutions of the meter (N_s) are counted. This experiment is repeated over a complete range of velocities and a best fitlinear relation is developed.

Float, types, velocity rod

Discharge measurement by floats
Floats are used to measure velocity for a small stream in flood, small stream with rapidly changing water surface and for preliminary analysis.

$$V_s = \frac{L}{t}$$

V_s = surface velocity L = Distance travelled
t = time taken to travel the float

Discharge = V_{av} x A
V_{av} = average velocity = 0.85 to 0.95 times surface velocityA = cross-sectional area

Types of floats
Surface float: a simple float moving on stream surface, wooden or metallic object, leaf, orange
Subsurface float: two floats tied together by thin cord, one float submerged
Rod float: cylindrical rod partly submerged

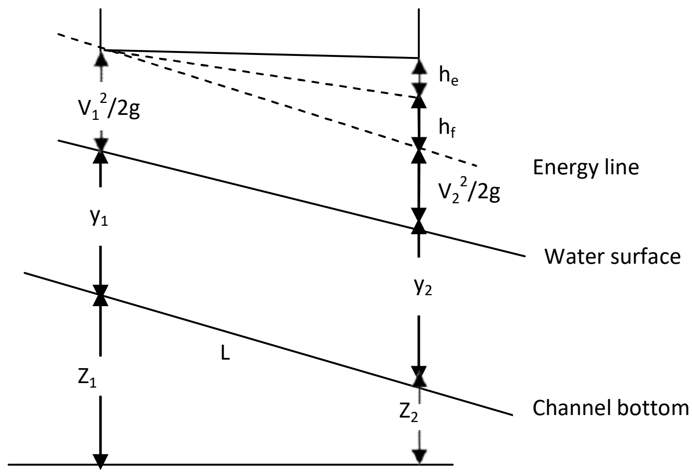
Method
Select straight reach free from current and eddies.
Measure distance between upstream and downstream section.
Divide the cross-section into a number of subsections.

For each subsection, release float at an upstream section and note the time taken by float to reach downstream section. Find average velocity of different sections and compute discharge.

Slope area method

This is indirect method of discharge measurement. In this method, Manning's equationand Bernoulli's equation are used to estimate the discharge for high floods based on previous flood marks. Two sections along a river reach are selected. The cross-sectional area of each section and the longitudinal profile between the sections is measured.

1 2



Datum

Fig. 4.8: Slope-area method

By using Bernoulli's equation for sections 1 and 2

$$Z_1 + y_1 + \frac{V_1^2}{2g} = Z_2 + y_2 + \frac{V_2^2}{2g} + h_L$$

Z_1, Z_2 : Datum headat sections1 and 2y_1, y_2 : water depthat sections1 and 2 V_1, V_2: velocities at sections1 and 2 h_L = Head loss

$h_L = h_f + h_e$

h_f = Frictional lossh_e = eddy loss

Denoting Z + y = h = water surface elevation above the datum

$$h_1 + \frac{V_1^2}{2g} = h_2 + \frac{V_2^2}{2g} + h_f + h_e$$

$$h_f = (h_1 - h_2) + \left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right) - h_e \qquad \text{(a)}$$

From Manning's equation,

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

Q =Discharge
n = Roughness coefficientA = Cross-sectional area
R = Hydraulic radius = A/P where P = wetted perimeterS_f= Slope of energy line between two points
In other form, Manning's equation is expressed as

$Q = K\sqrt{S_f}$ (b)

K = Conveyance of channel = $\frac{AR^{2/3}}{n}$

$S_f = \frac{h_f}{L}$

For two sections, average conveyance is

$K = \sqrt{K1K2}$ (c)

Where $K1 = \frac{A_1R_1^{2/3}}{n1}$ and $K2 = \frac{A_2R_2^{2/3}}{n2}$

Eddy loss is given by

$h_e = K_e \left|\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right|$ where K_e = Eddy loss coefficient

Procedure to compute peak discharge by using slope-area method
Compute cross-sectional area, wetted perimeter, hydraulic radius, K1and K2 at section 1 and 2.Compute K using $K = \sqrt{K1K2}$.
For first iteration, assume $V_1 = V_2$. This leads to $h_f = h_1 - h_2$ = Fall in water surface between two sections. So take $h_f = h_1 - h_2$.
Calculate Q using eq.$Q = K\sqrt{S_f} = K\sqrt{h_f/L}$.
Compute $V_1(= Q/A_1)$ and $V_2(= Q/A_2)$.

Now calculate a refined value of h_f by using eq.$(h_f)refined = (h_1 - h_2) + (\frac{V_1^2}{2g} - \frac{V_2^2}{2g}) - h_e$.
Take refined value of h_f for next iteration and repeat steps 3 to 5until the difference between twosuccessive values of h_f is negligible.
Compute Q using final value of h_f.

Recommended criteria
Distance between two sections = 75 times flood depth
Fall in water head>15 cm
Straight and uniform reach
Quality of high-water marks should be good.

Difference between slope-area and velocity area method
Velocity-Area method is a direct method for discharge measurement, whereas slope area method is an indirect method of discharge measurement
In velocity-Area method, measurement is performed across a cross-section. The cross-section of a riveris divided into a number of subsections by imaginary verticals. The depth of flow and velocity is measured at each vertical.
In slope area method, two sections along a river reach are selected. The cross-sectional area of each section and the longitudinal profile of high flood line between the sections is measured.

In velocity-Area method, velocity is measured by current meter or floats. In slope area method, velocity is computed from the concept of Hydraulics.
In slope area method, the segmental discharge is obtained by multiplying segmental area and mean velocity, and the total discharge is obtained by summing the segmental discharge.

In slope area method, the computation of discharge is based on Manning's equation and Bernoulli'sequation. No trial and error is needed in velocity area method, whereas h_f is obtained by trial and error approachin slope area method.

Flow measuring structures

These structures produce unique control section in the flow. For such structures,Q =f(H)

Q= discharge

H = Water surface elevation measured from a specified datum Free flow: flow independent of downstream water level

Submerged or drowned flow: flow affected by downstream water level

Various structures

Thin plate structures: made of metal plate, e.g. V-notch, rectangular notch

Long base weirs (Broad-crested): made of concrete or masonry

Flume: Channel having constriction

Formula

Rectangular notch: $Q = \frac{2}{3} C_d \sqrt{2g} . L H^{1.5}$

V-notch: $Q = \frac{8}{15} C_d \sqrt{2g} Tan \frac{\theta}{2} H^{5/2}$

Broad-crested weir: $Q = C_d Lh\sqrt{2g(H-h)}$ and $Q_{max} = 1.705 C_d LH^{3/2}$

Rating Curve (Stage-discharge relationship)

The relationship between discharge (Q) and stage (H) is known as rating curve. Continuous measurement of discharge is not feasible as it is costly and time consuming. So, discharge data with corresponding stage is collected from time to time as sample data, and a relation between stage and discharge (rating curve) is prepared from the sample data. As it is easy and inexpensive, stage is measured continuously. The rating curve is used to convert the measured stage into discharge. In this way, the continuous discharge value is obtained.

Q

Fig. 4.9: Rating curve

Equation of rating curve or stage-discharge relation is expressed in following exponential form.

$Q = a(H - H_0)^b$

Q = Discharge a, b = ConstantsH = Stage

H_0 = Stage for zero flow

Converting this equation to logarithmic form gives simple linear equation, which is then easy to use for further analysis.

$log Q = b log(H - H_0) + log a$

Or, Y=bX+c

Y =logQ, X = log(H-H_0)

The rating curve remains valid so long as the condition at a site remains stable.

Determination of parameters of rating curveDetermination of H_0

Graphical approach

Plot H and Q on a plain graph and draw best fit curve. Extrapolate the curve backwards to touch ordinate axis where Q =0. Take H corresponding to Q = 0 as H_0. Plot Q and H-Ho in log scale and check whether the plot is straight line. If not, take another value of H_0 close to it and repeat the procedure until straight line plot is obtained.

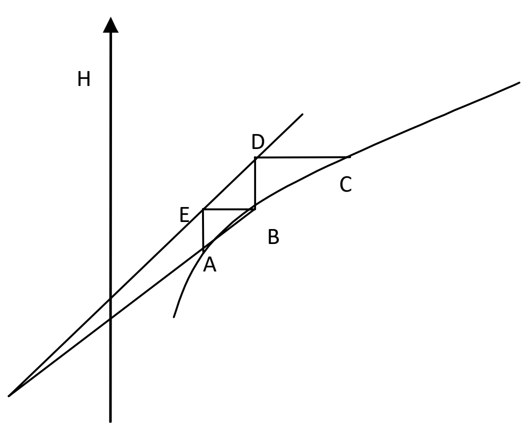
Plot Q vsH to an arithmetic scale and fit the smooth curve. Select three points on the curve such that their discharges are in geometric progression.

i.e.$Q2 = \sqrt{Q1Q3}$

Note H1, H2 and H3corresponding to Q1, Q2 and Q3. Compute H_0 by

$$H_0 = \frac{H1H3 - H2^2}{H1 + H3 - 2H2}$$

Plot Q vsH to an arithmetic scale and fit the smooth curve. Select three points A, B, and C on the curve such that their discharges are in geometric progression.Draw vertical lines at A and B and horizontal linesat B and C. Then two straight lines ED and BA are drawn to intersect at F as show in figure. The ordinate of F is required value of H_0.



F
Q

Determine a, b and H_0 simultaneously by least square optimization method.

Value of a and b from regression

$$b = \frac{N(\sum XY) - (\sum X)(\sum Y)}{N(\sum X^2) - (\sum X)^2}$$

$$c = \frac{\sum Y - b(\sum X)}{N}$$

a =10^c

Coefficient of correlation

$$r = \frac{N(\sum XY) - (\sum X)(\sum Y)}{\sqrt{N(\sum X^2) - (\sum X)^2}\sqrt{N(\sum Y^2) - (\sum Y)^2}}$$

Methods for extension of rating curve

Extension based on Logarithmic plotting of rating curve or using the rating equation
Velocity area method: Extend stage-velocity and stage-area curve.
Conveyance slope method based on Manning equation:
Flood marks in the river course provides water surface slope of the peak.

$$Q = \frac{1}{n} AR^{2/3} S^{1/2} = K\sqrt{S}$$

K = Conveyance of channel = $\frac{AR^{2/3}}{n}$

For different stages, compute K.

Compute S for different stages by using $S = \frac{Q^2}{K^2}$ from available Q-h data.

Plot and extend stage-conveyance(K) and stage-slope (S) curve. For different stage, take K and S from the two curves and compute Q as $K\sqrt{S}$.
Assumptions: For higher stages, the slope remains constant

Control

Control is combined effect of channel and flow parameters, which govern the stage-dischargerelationship.

If the rating curve does not change with time, the control is called permanent control. In other words, the station with permanent control has single valued rating curve.
If the rating curve changes with time, it is called shifting control.

Shifting controls
Vegetation growth, dredging or channel encroachment: no unique rating curve
Aggradation or degradation in alluvial channel: no unique rating curve
Variable backwater effect: same stage indicating different discharges
Unsteady flow effects of rapidly changing stage: for the same stage, low discharge during risingand high discharge during falling (looped rating)

Correction for backwater effect: To take into account the backwater effect, secondary gauge is installed at some distance downstream of gauging site and the readings of both gauges are taken. Then, the fall of water surface in the reach is computed. The relationship for actual discharge (Q) is given by

$$\frac{Q}{Q_0} = \left(\frac{F}{F_0}\right)^m$$

Q_0 = Normalized discharge at the given stage when fall = F_0, when the stage in the river is same in bothcases
F = Actual fall
m = exponent ≈0.5

Unsteady flow correction: Correction has to be applied in case of unsteady flow due to flood wave. The actual discharge (Q) under unsteady condition is given by

$$Q = Q_0\sqrt{1 + \frac{1}{V_w S_0}\frac{dh}{dt}}$$

Q_0 = discharge under steady flow conditions V_w = Velocity of flood wave
S_0 = Bed slope of river
dh/dt = rate of change of stage

4.11 Estimating mean monthly flow for ungauged basin of Nepal

Medium irrigation project (MIP) Method
The MIP method presents a technique for estimating the distribution of monthly flows throughout a year for ungauged locations. For application to ungauged sites, it is necessary to obtain one flow measurementin the low flow period from November to April.
In the MIP Method, Nepal has been divided hydrologically into seven zones. Once the catchmentarea of the scheme, one flow measurement in the low flow period and the hydrological zone is identified, long-term average monthly flows can be determined by multiplying the unit hydrograph (of the concerned region) with the measured catchment area.
Hydrological zone can be identified based on the location of the scheme in the hydrologically zoned map of Nepal.
For catchment areas less than 100 km^2, MIP method is used for better results.

If the measured date is on 15th of the particular month, the coefficient given in the table is directly used. For other date of measurement, coefficient for that date is found by interpolation.

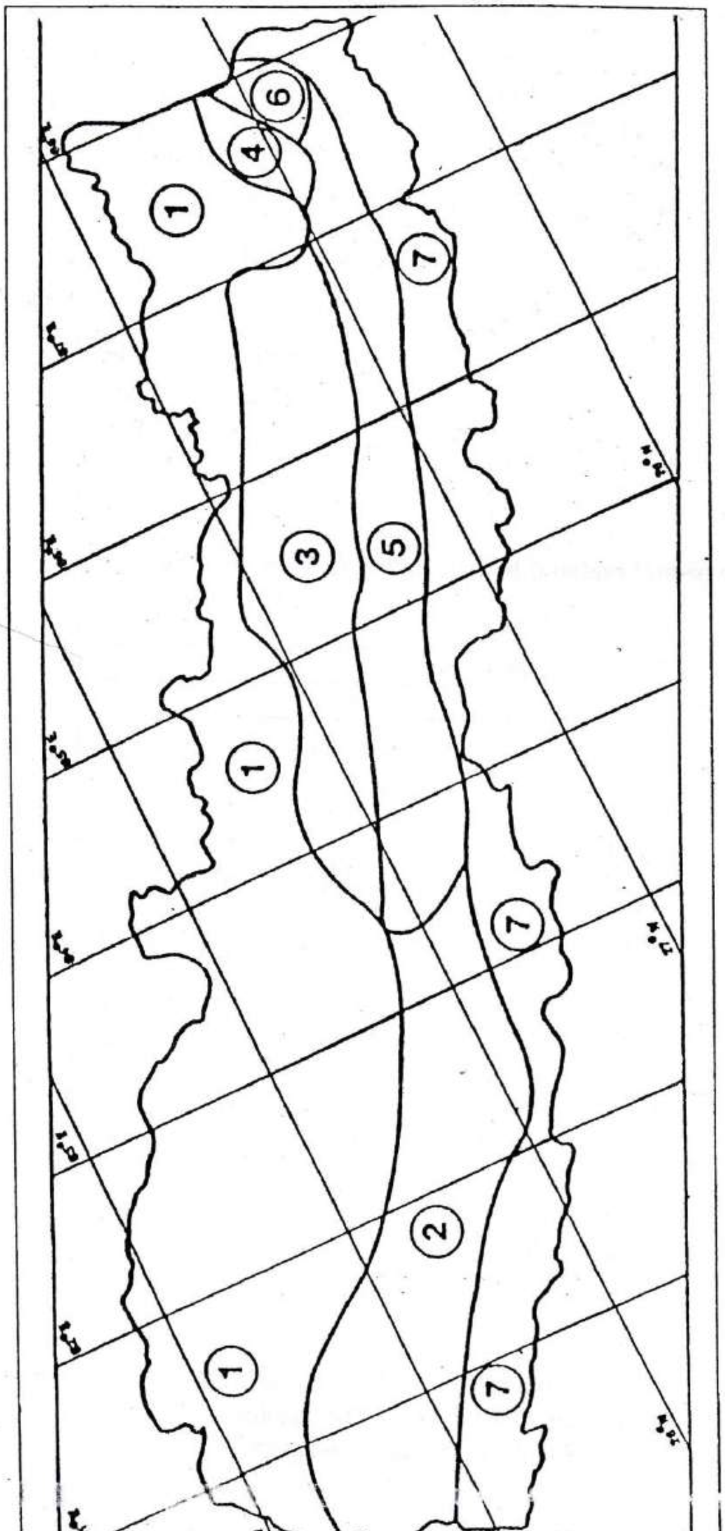
$$April\ flow = \frac{1}{coefficient\ of\ a\ particular\ month}\ Measured\ discharge$$

Monthly flow = April flow x Monthly coefficient

MIP non-dimensional regional hydrographs (Coefficient)

Month	Region						
	1	2	3	4	5	6	7
May	2.60	1.21	1.88	2.19	0.91	2.57	3.50
Jun	6.00	7.27	3.13	3.75	2.73	6.08	6.00
Jul	14.50	18.18	13.54	6.89	11.21	24.32	14.00
Aug	25.00	27.27	25.00	27.27	13.94	33.78	35.00
Sep	16.50	20.19	20.83	20.91	10.00	27.03	24.00
Oct	8.00	9.09	10.42	6.89	6.52	6.08	12.00
Nov	4.10	3.94	5.00	5.00	4.55	3.38	7.50
Dec	3.10	3.03	3.75	3.44	3.33	2.57	5.00
Jan	2.40	2.24	2.71	2.59	2.42	2.03	3.30
Feb	1.80	1.70	1.88	1.88	1.82	1.62	2.20
Mar	1.30	1.33	1.38	1.38	1.36	1.27	1.40
Apr	1.00	1.00	1.00	1.00	1.00	1.00	1.00

Note: all values for mid month



1. Mountain catchments.
2. Hills to the north of Mahabarat, river rising north of Siwaliks, inner terai.
3. Pokhara, Nuwakot, Kathmandu, Sunkosi tributaries.
4. Lower Tamur Valley
5. Rivers draining Mahabarats.
6. Kankai Mai basin.
7. Rivers draining from Churia range to the terai.

Hydrological regions of Nepal for MIP method

WECS/DHM (Hydest) Method

It is developed for predicting river flows for catchment areas larger than 100 km^2 of ungauged rivers based on hydrological theories, empirical equations and statistics. For long term average monthly flows, all areas below 5000m are assumed to contribute flows equally per km^2 area.
The average monthly flows can be calculated by the equation:
$Q_{mean,(month)}$ = C x (Area of Basin)^{A1} x (Area below 5000m +1)^{A2} x (Mean Monsoon precipitation)^{A3}.
Where $Q_{mean(month)}$ is the mean flow for a particular month in m^3/s, C, A1, A2 and A3 are coefficients of the different months.
The catchment area can be calculated from the topographical maps (maps that show contours) once the intake location is identified.

The input data required in the equation are total basin area (km^2), basin area below 5000m (km^2) and the average monsoon precipitation (km^2) estimated from isohyetal map.

Values of coefficients for WECS/DHM method

Month	C	A1	A2	A3
Jan	0.01423	0	0.9777	0
Feb	0.01219	0	0.9766	0
Mar	0.009988	0	0.9948	0
Apr	0.007974	0	1.0435	0
May	0.008434	0	1.0898	0
Jun	0.006943	0.9968	0	0.2610
Jul	0.02123	0	1.0093	0.2523
Aug	0.02548	0	0.9963	0.2620
Sep	0.01677	0	0.9894	0.2878
Oct	0.009724	0	0.9880	0.2508
Nov	0.001760	0.9605	0	0.3910
Dec	0.001485	0.9536	0	0.3607

Catchment Area Ratio Method (CAR Method)
If the two catchments are hydrologically similar, then the extension of hydrological data for proposed site under study could be done simply by multiplying the available long term data at hydrologically similar catchments (HSC) with ratio of catchment areas of base (proposed site under study) and index (HSC) stations.

$$Q_b = Q_i \frac{A_b}{A_i}$$

Where, Q = discharge in m^3/s
A = drainage area in sq.km
Suffix 'b' stands for base station and i stands for index station.
This method is useful if the hydro-meteorological data of the index station having similar catchmentcharacteristics with the base station are available for the data extension.

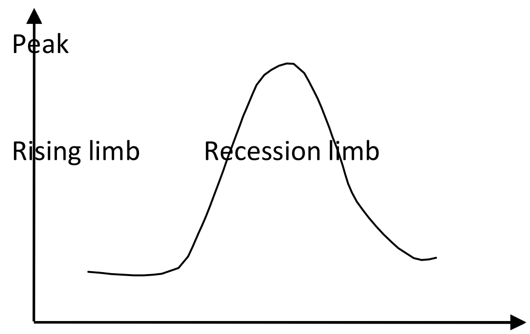
Chapter 5: Hydrograph analysis

Hydrograph

Hydrograph is a graphical plot of discharge (Q) of a river at a given location over time. It is the output or total response of a basin.

Components of hydrograph

Q



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Hydrograph

Rising limb

It is ascending portion of hydrograph. It is influenced by storm and basin characteristics. The rising limb rises slowly in the early stage of flood but more rapidly towards the end portion. This is because in the initial stage the losses are high. The flow begins to build up in the channel as the storm duration increases. It gradually reaches the peak when maximum area contributes.

Peak or crest segment

It is the part which contains peak flow, which is of interest to hydrologists. Peak of hydrograph occurs when all portions of basins contribute at the outlet simultaneously at the maximum rate. Depending upon the rainfall-basin characteristics, the peak may be sharp, flat or may have several well defined peaks.

Recession limb

Recession limb represents withdrawal of water from the storage built up in the basin during the earlier phase of the hydrograph. It extends from the point of inflection at the end of the crest to the beginning of natural groundwater flow. The recession limb is affected by basin characteristics only and independent of the storm.

Equation for recession curve

$$Q_t = Q_0 K_r^t$$

Q_0: initial discharge

Q_t: discharge at a time interval of t days K_r: recession constant

Alternative form

$$Q_t = Q_0 e^{-at}$$

Where a = $-\ln K_r$

Terms

Time to peak: time lapse between starting of the rising limb to the peak

Time lag: time interval between centre of mass of rainfall hyetograph to the centre of mass of runoffhydrograph.

Time of concentration: time taken by a drop of water to travel from the remotest part to the outlet

Time base of hydrograph: time between starting of runoff hydrograph to the end of direct runoff due tostorm.

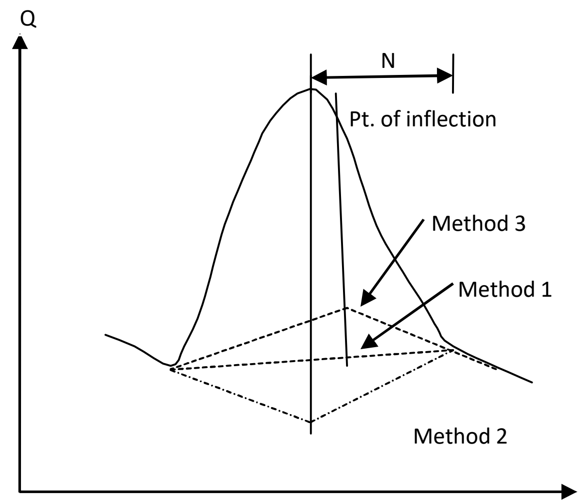
Direct runoff and base flowDirect runoff

It is the part of precipitation which appears quickly as flow in the river. (direct runoff = surface +subsurface)

Base flow

The part of runoff which receives water from the groundwater storage is called base flow.

Base flow separation



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Base flow separation methods

Straight line method

Join the beginning of surface runoff to a point on the recession limb representing the end of directrunoff.

End point: by expert judgment or empirical equationEmpirical equation to find end of direct runoff

$N = 0.83\ A^{0.2}$

N = time interval from the peak to the end of direct runoffA = Basin area

Extend the base flow curve prior to the commencement of surface runoff till it intersects the ordinate drawn at the peak point. Join this point to the end point of direct runoff

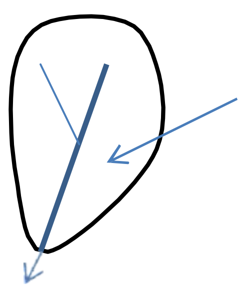
Extend the base flow recession curve backwards after the depletion of flood water till it intersects the ordinate at the point of inflection. Join this point to the beginning of the surface runoff by smooth curve.
Direct runoff hydrograph: the surface runoff hydrograph obtained after separating base flowTypes of stream
Perennial: always carry flow
Intermittent: limited contribution from groundwater
Ephemeral: no base flow

Yield: total quantity of water that can be expected from a stream in a given period.

Unit Hydrograph

A unit hydrograph (UH) of a basin is defined as a direct runoff hydrograph (DRH) resulting from one unit depth of rainfall excess generated uniformly over the basin at a constant rate for an effective duration (D). The term unit refers to a unit depth of rainfall excess which is 1cm in SI unit and 1 inch in FPS unit. (Rainfall excess/effective rainfall = rainfall-loss)

Rainfall excess of
1cm for D hour



UH

Duration of unit hydrograph (D-hour UH): duration of rainfall excessAssumptions
Constant intensity of excess rainfall within the effective
Uniform distribution of excess rainfall over the basin
Constant base time of the DRH for excess rainfall of given duration
Linear model: principle of superposition and proportionality holds
Principle of time invariance holds
-Given excess rainfall will always produce the same DRH whatever may be the season of the year (unchanging basin characteristics)

Principles applied in UH

Linearity principles
Linear relationship means output varies linearly with input. This principle is expressed by convolutiontheorem.

If $I(r)$ is intensity of input at time r and $u(t-r)$ is the unit response after time t, then total response isgiven by $Q(t) = \int_0^t I(r)\,u(t-r)dr$. This is convolution integral.

There are two principles of linearity.
Principle of proportionality: If a solution y is multiplied by a constant c, the resulting function cy is alsoa solution.
r_e = excess rainfall, UH = Unit hydrograph (solution)
Output (DRH) = r_e*UH
Principle of superposition: If two solutions y1 and y2 of the equation are added, the resulting functiony1+y2 is also a solution of the equation.
r_{e1}, r_{e1} = excess rainfall at t hr interval, UH = Unit hydrograph (solution)Output (DRH) = (r_{e1}*UH)+ (r_{e2}*UH lagged by t hr)

Principle of time invariance: Given excess rainfall will always produce the same DRH whatever may bethe season of the year (unchanging basin characteristics)

Features
Rainfall excess (r_e) =1cm, runoff depth (r_d) = 1cm
Continuity: Total depth of rainfall excess = total depth of direct runoff
Runoff volume (V_d) = Basin area(A) x r_d = Ax1cm
Rainfall intensity: 1/D in cm/h
Lumped response: catchment as a single unit
Initial loss absorbed by basin, no effect of antecedent storm condition

Applications of UH
Computation of flood hydrograph for the design of hydraulic structures
Extension of flow records at a site
Flood forecasting
Comparing the basin characteristics

Limitations of UH
Minimum basin size> 2km^2, Maximum basin size up to 5000 km^2
Not suitable for very long basins
Applicable for short duration
Not very suitable for basins having large snow cover
UH is not applicable for basins having large storages
UH is not applicable for basins having high variation of rainfall intensity.
Derivation of unit hydrograph Selection criteria for flood hydrograph
Selection of isolated storms occurring individually
Fairly uniform rainfall over the entire basin
Duration of rainfall: 1/5 to 1/3 of basin lag
Range of rainfall excess: 1 to 4 cm

Derivation of UH for single storm
Given: streamflow data (Q) and basin area (A)
Single storm: all of the rainfall excess occurs at a reasonably uniform rate over a fairly short time period
Separate baseflow (BF).
DRH=Q-BF
Volume of DRH (V_d) = $\sum DRH * \Delta t$
CRunoff depth (r_d) = V_d/A
UH= DRH/r_d
Effective duration of UH = Duration of excess rainfall. Check whether total depth of runoff = total rainfall excess

Derivation of UH for multiple storms
Multiple storms: relatively long and varying intensities if rainfall
Storms: divided into number of equal periods and fairly constant rate of rainfall for each period Duration of UH = Duration of period of each storm

De-convolution method
Given: DRH data and rainfall excess data
(If DRH is not given, compute base flow and compute DRH by subtracting baseflow from streamflowdata)
Convolution Equation in discrete form

$$Q_n = \sum_{m=1}^{n\leq m} P_m U_{n-m+1}$$

n = number of runoff ordinates
m = number of periods of rainfall excessQ_n = Direct runoff
P_m= Excess rainfall U_{n-m+1} = UH ordinate

Use above equation for computing ordinate of UH with excess rainfall and direct runoff data. For complex multi-peaked hydrograph: solution of above equation by least square regression.

Computation of runoff from given UH

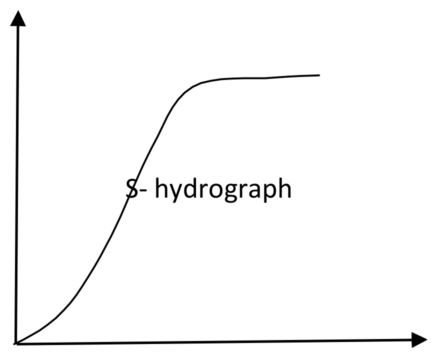
Single storm
DRH =UH * rainfall excess.
Total runoff=DRH+BF

Multiple storms
Given: UH and effective rainfall for multiple durationsUse principle of proportionality and superposition
DRH1=UH * first rainfall excess.
DRH2=UH*second rainfall excess lagged by duration of first and second rainfallDRH3 = UH*third rainfall excess lagged by duration of first and third rainfall
So on….
DRH= DRH1+DHR2+DHR3+……
Total runoff=DRH+BF

S-Hydrograph

S Hydrograph is a hydrograph resulting from a continuous excess rainfall at a constant rate of 1cm/h for an indefinite period. It is a theoretical concept. The curve is named S hydrograph as it looks like deformed S shape. The curve is obtained by adding a series of D-h unit hydrographs spaced at D-h apart.

Q



t

The S-curve reaches a maximum equilibrium discharge at a time equal to the time base of the first unithydrograph.
Unit rainfall excess = 1 cm in D hrRainfall intensity = 1/D in cm/hr
If A = basin area in km^2 and D is in hour, then
Equilibrium discharge $(Q_s) = \left(\frac{1}{D} x \frac{1}{3600 x 100}\right)(A x 10^6) = 2.778 \frac{A}{D}$ m^3/s

Construction of S-curveU(t) = S(t) – S(t-D)
S(t) = U(t)+ S(t-D)
where D = Duration of UH, S(t) = ordinate of S-curve at t, U(t) = ordinate of UH at t, S(t-D) = ordinate ofS-curve at t-D

In other words,
Ordinate of S-curve at t = ordinate of D-hr UH at t + S-curve addition at time tFor $t \leq D$, S(t-D) = 0.

Computation of Unit hydrograph of different durations

In the computation of flood hydrograph, if the duration (D) of given UH and the duration (D’) of excessrainfall is different, then the UH of D hour should be converted to UH of D’ hour.

Given: UH of duration D
To compute: UH of duration D’n = D’/D
If n is integer, use superposition method or S-curve method.If n is real, use S-curve method.

Superposition method
Lag the UH ordinate by D, 2D, ….(n-1)D.
U1=Sum of the ordinates of all UHs.
Ordinate of D'-hour UH = U1/n

S Hydrograph method
Compute S-curve addition (=S(t-D)).
Compute the ordinate of S-curve.S1 = UH(t)+ S(t-D)
Lag the ordinates of S1 hydrograph by the duration D'. This is S2.
Ordinate of D'-hour UH= (S1-S2)/n

In case of D'<D and the time interval of data is not equal to D', first plot the given UH and read thevalues with time interval equal to D'. Then follow above steps.

If the ordinates of UH becomes negative or shows fluctuations in the tail part, then manually smoothenthe tail part.

Basic Numericals of Unit Hydrograph

Derivation of UHSingle storm

Given below are the observed flows from a storm of 4hr duration on a stream with a catchment area of613 km^2. Derive 4hr unit hydrograph. Make suitable assumptions regarding base flow.

Time (hr)	0	4	8	12	16	20	24	28	32	36	40	44	48
Observed flow (m3/s)	10	110	225	180	130	100	70	60	50	35	25	15	10

Solution:

Catchment area (A) = 613 km^2 Assume base flow (BF) = 10 m^3/sDirect runoff (Q_{dr}) = Q-BF Volume of runoff (V) = $\sum Q_{dr}$ Δt Runoff depth (r_d) = V/A

Divide Q_{dr} by r_d to get UH ordinate.

Δt is same for each runoff ordinate.

Δt = 4 hour = 4x3600 s

V = $\sum Q_{dr}\ \Delta t = \Delta t \sum Q_{dr}$ = 890x4x3600

$$r_d = \frac{V}{A} = \frac{890\times4\times3600}{613\times10^6} = 0.02\text{m} = 2\text{cm}$$

Time (hr)	0	4	8	12	16	20	24	28	32	36	40	44	48
Q (m^3/s)	10	110	225	180	130	100	70	60	50	35	25	15	10
BF (m^3/s)	10	10	10	10	10	10	10	10	10	10	10	10	10
Qdr (m^3/s)	0	100	215	170	120	90	60	50	40	25	15	5	0
UH (m^3/s)	0	50	108	85	60	45	30	25	20	13	7.5	2.5	0

The ordinates of a hydrograph of a surface runoff (DRH) resulting from 4.5cm of rainfall excess ofduration 8hr in a catchment are as follows:

Time (hr)	0	5	13	21	28	32	35	41	45	55	61	91	98	115	138
Discharge (m^3/s)	0	40	210	400	600	820	1150	1440	1510	1420	1190	650	520	290	0

Derive the ordinates of 8hr-unit hydrograph.Solution:

Direct runoff = Q

Rainfall excess (R_e) = 4.5cm

For single storm, UH ordinate = Q/R_e

Time (hr)	0	5	13	21	28	32	35	41	45	55	61	91	98	115	138

Discharge (m^3/s)	0	40	210	400	600	820	1150	1440	1510	1420	1190	650	520	290	0
UH	0	8.9	46.7	88.9	133.3	182.2	256	320	335.6	316	264.4	144	116	64.44	0

<u>Multiple storm</u>

The following table gives the ordinates of a DRH resulting from two successive 3-hour durations of rainfall excess value of 2 cm and 4 cm respectively.

t (hr)	0	3	6	9	12	15	18	21	24	27	30
DRH (m^3/s)	0	120	480	660	460	260	160	100	50	20	0

Derive the ordinates of 3-hr UH.Solution:
a. Effective rainfall, R_1 = 2 cm and R_2 = 4cm
It is a case of multiple storms. We have to use discrete time convolution equation to compute UH ordinate. The equation is

$$Q_n = \sum_{m=1}^{n\le m} R_m U_{n-m+1}$$

Q = Direct runoff, R = Excess rainfall, U = UH ordinate
Here, total no. of runoff ordinates (n) = 9 Total number of rainfall excess values (m) =2

For n =1, m =1$Q_1 = R_1U_1$
$U_1 = Q_1/R_1$ =120/2 = 60
For n = 2, m=1, 2$Q_2 = R_1U_2+R_2U_1$
$U_2 = (Q_2-R_2U_1)/R_1$ = (480-4x60)/2 = 120
For n = 3 onwards, m= 1, 2. So, we can use the similar expression as that of U_2 for n = 3 onwards.$U_n = (Q_n-R_2U_{n-1})/R_1$

$U_3 = (Q_3-R_2U_2)/R_1$ = (660-4x120)/2 = 90$U_4 = (Q_4-R_2U_3)/R_1$ = (460-4x90)/2 = 50 $U_5 = (Q_5-R_2U_4)/R_1$ = (260-4x50)/2 = 30 $U_6 = (Q_6-R_2U_5)/R_1$ = (160-4x30)/2 = 20 $U_7 = (Q_7-R_2U_6)/R_1$ = (100-4x20)/2 = 10 $U_8 = (Q_8-R_2U_7)/R_1$ = (50-4x10)/2 = 5 $U_9= (Q_9-R_2U_8)/R_1$ = (20-4x5)/2 = 0

Resulting UH

t (hr)	0	3	6	9	12	15	18	21	24	27	30
UH(m^3/s)	0	60	120	90	50	30	20	10	5	0	0

UH to flood hydrograph

The ordinate of a 4-h UH of a catchment of area 1000km^2 are given below. Calculate flood hydrograph resulting from two successive 4-h storms having rainfall of 1.5cm each. Assume uniform base flow of 10 m^3/s and ϕ-index equal to 0.10 cm/hr.

t(hr)	0	4	8	12	16	20	24	28	32	36	40	44
4hr UH (m^3/s)	0	20	60	150	120	90	66	50	32	20	10	0

Solution:

ϕ-index (infiltration loss) = 0.1 cm/hrFor 4 hour, loss (L) = 4x0.1 =0.4 cm

Rainfall values, R1 = 1.5 cm and R2 = 1.5 cmRainfall excess (R_{e1}) = R1-L = 1.5-0.4 =1.1cmRainfall excess (R_{e2}) = R2-L = 1.5-0.4 =1.1cm

DHR1 = UHxR_{e1}

DRH2 =UHxR_{e2} (lagged by 4 hour)

DRH = DRH1+DRH2Q = DRH + BF

Computation of flood hydrograph

t(h)	4 hr UH (m^3/s)	DRH1 (m^3/s)	DRH2 (m^3/s)	DRH (m^3/s)	BF (m^3/s)	Q (m^3/s)
0	0	0		0	10	10
4	20	22	0	22	10	32
8	60	66	22	88	10	98
12	150	165	66	231	10	241
16	120	132	165	297	10	307
20	90	99	132	231	10	241
24	66	72.6	99	171.6	10	181.6
28	50	55	72.6	127.6	10	137.6
32	32	35.2	55	90.2	10	100.2
36	20	22	35.2	57.2	10	67.2
40	10	11	22	33	10	43
44	0	0	11	11	10	21
(48)			0	0	10	10

UH of different durations

The ordinates of a 4 hour UH of a basin of area 25 km^2 are given below.

t (hr)	0	4	8	12	16	20	24	28	32	36	40	44	48	52
UH(m^3/s)	0	30	55	90	130	170	180	160	110	60	35	20	8	0

Calculate the following.

4-hr DRH for a rainfall of 3.25cm with ϕ-index of 0.25cm.

a 12-hr UH by using the method of superposition.

a 12-hr UH by using the S-curve method.

Solution:

Rainfall (R) = 3.25 cm, ϕ-index = 0.25cmRainfall excess (re) = 3.25-0.25 = 3cm

DRH = UH xre Computation of DRH

t (hr)	0	4	8	12	16	20	24	28	32	36	40	44	48	52
UH(m^3/s)	0	30	55	90	130	170	180	160	110	60	35	20	8	0
DRH (m^3/s)	0	90	165	270	390	510	540	480	330	180	105	60	24	0

Required duration of UH (D') =12 hrGiven duration (D)=4 hr

n = D'/D = 3 (integer)

UHa = UH lagged by 4 hr, UHb = UH lagged by 8 hourUH1 = UH + UHa+ UHb

12hr-UH = UH1/(D'/D) = UH1/3

Computation of 12-hr UH using method of superposition

t (hr)	UH	Uha	Uhb	UH1	12-hr UH (m^3/s)
0	0			0	0
4	30	0		30	10
8	55	30	0	85	28.3
12	90	55	30	175	58.3
16	130	90	55	275	91.7
20	170	130	90	390	130
24	180	170	130	480	160
28	160	180	170	510	170
32	110	160	180	450	150
36	60	110	160	330	110
40	35	60	110	205	68.3
44	20	35	60	115	38.3
48	8	20	35	63	21
52	0	8	20	28	9.3

56		0	8	8	2.7
60			0	0	0

c. S-curve addition = Ordinate of S-curve at (t-D)
Ordinate of S curve (S1) = ordinate of UH+ S-curve additionS 2 = S1 lagged by 12 hr hour
12hr-UH = (S1-S2)/(D'/D) = (S1-S2)/3

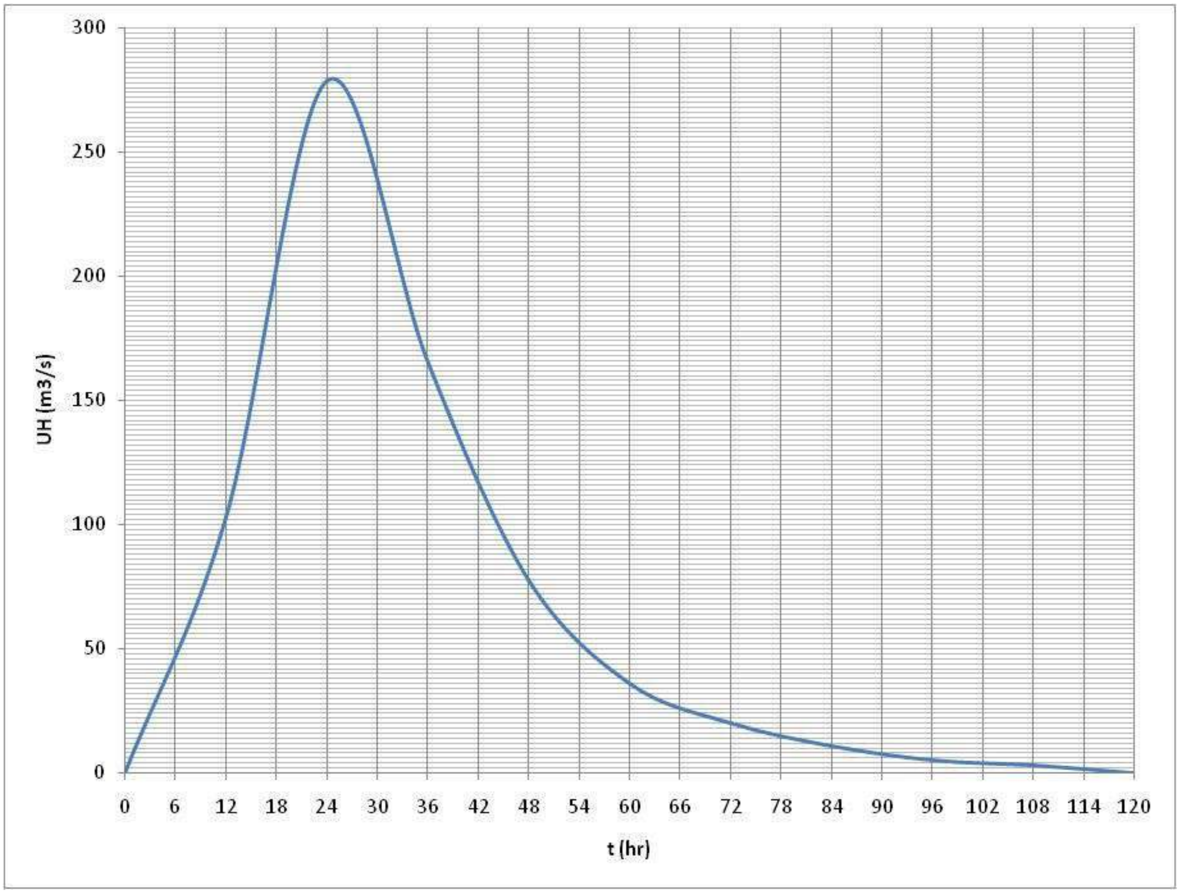
Computation of 12-hr UH using S-Curve method

t (hr)	UH	S-Curve addition	S-curve (S1)	S2	12-hr UH (m^3/s)
0	0		0		0
4	30	0	30		10
8	55	30	85		28.3
12	90	85	175	0	58.3
16	130	175	305	30	91.7
20	170	305	475	85	130
24	180	475	655	175	160
28	160	655	815	305	170
32	110	815	925	475	150
36	60	925	985	655	110
40	35	985	1020	815	68.3
44	20	1020	1040	925	38.3
48	8	1040	1048	985	21
52	0	1048	1048	1020	9.3
56			1048	1040	2.7
60			1048	1048	0
64			1048	1048	0

Given below is a 12-hr UH. Derive 6-hr UH.

t (hr)	0	12	24	36	48	60	72	84	96	108	120
UH(m^3/s)	0	103	279	165	78	36	20	11	5	3	0

Solution:
Required duration of UH (D') =6 hrGiven duration (D) =12 hr
n = D'/D = 0.5 (real)
Here, D'<D. To derive UH of 6 hr, the interval of ordinates of given UH should be at least 6hour.Plot given UH versus t on a graph paper and get the values of UH at 6 hour interval.



S-curve addition = Ordinate of S-curve at (t-D)
Ordinate of S curve (S1) = ordinate of UH+ S-curve additionS2 = S1 lagged by 6 hour
6-hr UH = (S1-S2)/(D'/D) = (S1-S2)/0.5

Computation of 6-hr UH

t (hr)	UH(m3/s)	S curve addition	S1	S2	6-hr UH	6-hr UH (corrected)
0	0		0		0	0
6	48		48	0	96	96
12	103	0	103	48	110	110
18	191	48	239	103	272	272
24	279	103	382	239	286	286
30	238	239	477	382	190	190
36	165	382	547	477	140	140
42	117	477	594	547	94	94
48	78	547	625	594	62	62
54	53	594	647	625	44	44
60	36	625	661	647	28	28

66	27	647	674	661	26	26
72	20	661	681	674	14	14
78	15	674	689	681	16	10
84	11	681	692	689	6	6
90	8	689	697	692	10	4
96	5	692	697	697	0	0
102	4	697	701	697	8	0
108	3	697	700	701	-2	0
114	2	701	703	700	6	0
120	0	700	700	703	-6	0
126		703	700	700	0	0

The UH of 6 hour should be corrected manually from 90 hour onwards to make it smooth.

3.1.5 Water availability for diversion crop water requirement; and water requirement at headwork.

SOIL–WATER RELATIONS AND IRRIGATION METHODS

SOILS

Soil mainly consists of finely divided organic matter and minerals (formed due to disintegration of rocks). It holds the plants upright, stores water for plant use, supplies nutrients to the plants and helps in aeration. Soils can be classified in many ways, such as on the basis of size (gravel, sand, silt, clay, etc.), geological process of formation, and so on. Based on their process of formation (or origin), they can be classified into the following categories:

Residual soils: Disintegration of natural rocks due to the action of air, moisture, frost, and vegetation results in residual soils.

Alluvial soils: Sediment material deposited in bodies of water, deltas, and along the banks of the overflowing streams forms alluvial soils.

Aeolian soils: These soils are deposited by wind action.

Glacial soils: These soils are the products of glacial erosion.

Colluvial soils: These are formed by deposition at foothills due to rain wash.

Volcanic soil: These are formed due to volcanic eruptions and are commonly called as volcanic wash.

The soils commonly found in India can be classified as follows:

Alluvial Soils: Alluvial soils include the deltaic alluvium, calcareous alluvial soils, coastal alluvium, and coastal sands. This is the largest and most important soil group of India.

The main features of the alluvial soils of India are derived from the deposition caused by rivers of the Indus, the Ganges, and the Brahmaputra systems. These rivers bring with them the products of weathering of rocks constituting the mountains in various degrees of fineness and deposit them as they traverse the plains. These soils vary from drift sand to loams and from fine silts to stiff clays. Such soils are very fertile and, hence, large irrigation schemes in areas of such soils are feasible. However, the irrigation structures themselves would requirestrong foundation.

Black Soils: The black soils vary in depth from a thin layer to a thick stratum. The typical soil derived from the Deccan trap is black cotton soil. It is common in Maharashtra, western parts of Madhya Pradesh, parts of Andhra Pradesh, parts of Gujarat, and some partsof Tamil Nadu. These soils may vary from clay to loam and are also called heavy soils. Many black soil areas have a high degree of fertility but some, especially in the uplands, are rather poor. These are suitable for the cultivation of rice and sugarcane. Drainage is poor in such soils.

Red Soils: These are crystalline soils formed due to meteoric weathering of the an- cient crystalline rocks. Such soils are found in Tamil Nadu, Karnataka, Goa, south-eastern Maharashtra, eastern Andhra Pradesh, Madhya Pradesh, Orissa, Bihar, and some districts of West Bengal and Uttar Pradesh. Many of the so-called red soils of south India are not red. Redsoils have also been found under forest vegetation.

Lateritic Soils: Laterite is a formation peculiar to India and some other tropical countries. Laterite rock is composed of a mixture of the hydrated oxides of aluminium and iron with small amounts of manganese oxides. Under the monsoon conditions, the siliceous matterof the rocks is leached away almost completely during weathering. Laterites are found on the hills of Karnataka, Kerala, Madhya Pradesh, the estern Ghats of Orissa. Maharashtra, West Bengal, Tamil Nadu, and Assam.

Desert Soils: A large part of the arid region belonging to western Rajasthan, Haryana, and Punjab lying between the Indus river and the Aravalli range is affected by desert and conditions of geologically recent origin. This part is covered with a mantle of the blown sand which, combined with the arid climate, results in poor soil development. The Rajasthan desert is a vast sandy plain including isolated hills or rock outcrops at places. The soil in Rajasthan improves in fertility from west and north-west to east and north-east.

Forest Soils: These soils contain high percentage of organic and vegetable matterand are also called humus. These are found in forests and foothills.

Soils suitable for agriculture are called arable soils and other soils are non-arable.

Depending upon their degree of arability, these soils are further subdivided as follows:

Class I: The soils in class I have only a few limitations which restrict their use for cultivation. These soils are nearly level, deep, well-drained, and possess good water-holding capacity. They are fertile and suitable for intensive cropping.

Class II: These soils have some limitations which reduce the choice of crops and require moderate soil conservation practices to prevent deterioration, when cultivated.

Class III: These soils have severe limitations which reduce the choice of crops and require special soil conservation measures, when cultivated.

Class IV: These soils have very severe limitations which restrict the choice of cropsto only a few and require very careful management. The cultivation may be restricted to oncein three or four years.

Soils of type class I to class IV are called arable soils. Soils inferior to class IV are grouped as non-arable soils. Irrigation practices are greatly influenced by the soil characteristics. From agricultural considerations, the following soil characteristics are of particular significance.

Physical properties of soil,

Chemical properties of soil, and

Soil-water relationships.

PHYSICAL PROPERTIES OF SOIL

The permeability of soils with respect to air, water, and roots are as important to the growth of crop as an adequate supply of nutrients and water. The permeability of a soil depends on the porosity and the distribution of pore spaces which, in turn, are decided by the texture and structure of the soil.

Soil Texture

Soil texture is determined by the size of soil particles. Most soils contain a mixture of sand (particle size ranging from 0.05 to 1.00 mm in diameter), silt (0.002 to 0.05 mm) and clay (smaller than 0.002 mm). If the sand particles dominate in a soil, it is called sand and is a coarse-textured soil. When clay particles dominate, the soil is called clay and is a fine-textured soil. Loam soils (or simply loams) contain about equal amount of sand, silt, and clay and are medium-textured soils.

The texture of a soil affects the flow of water, aeration of soil, and the rate of chemical transformation all of which are important for plant life. The texture also determines the water holding capacity of the soil.

Soil Structure

Volume of space (i.e., the pores space) between the soil particles depends on the shape and size distribution of the particles. The pore space in irrigated soils may vary from 35 to 55 per cent. The term porosity is used to measure the pore space and is defined as the ratio of the volume of voids (i.e., air and water-filled space) to the total volume of soil (including water and air). The pore space directly affects the soil fertility (i.e., the productive value of soil) due to its influence upon the water-holding capacity and also on the movement of air, water, and roots through the soil.

Soils of uniform particle size have large spaces between the particles, whereas soils of varying particle sizes are closely packed and the space between the particles is less. The particles of a coarse-grained soil function separately but those of fine-grained soils function as granules. Each granule consists of many soil particles. Fine-textured soils offer a favourable soil structure permitting retention of water, proper movement of air and penetration of roots which is essential for the growth of a crop.

The granules are broken due to excessive irrigation, ploughing or working under too wet (puddling) or too dry conditions. Such working affects the soil structure adversely. The structure of the irrigated soil can be maintained and improved by proper irrigation practices some of which are as follows (1):

Ploughing up to below the compacted layers,

After ploughing, allowing sufficient time for soil and air to interact before preparing the seed bed or giving pre-planting irrigation,

The organic matter spent by the soil for previous crops should be returned in the form of fertilisers, manures, etc.,

Keeping cultivation and tillage operations to a minimum, and

Adopting a good crop rotation.

Green manures keep the soil fertility high. Crops like hamp, gwar, moong etc. are grown on the fields. When these plants start flowering, ploughing is carried out on the fields so that these plants are buried below the ground surface. Their decomposition makes up for the soildeficiencies.

The tendency of cultivators to grow only one type of crop (due to better returns) should be stopped as this cultivation practice leads to the deficiency in the soil of those nutrients which are needed by the crop. If the land is not used for cultivation for some season, the soil recoups its fertility. Alternatively, green manures can be used. Rotation of crops (which means growing different crops on a field by rotation) is also useful in maintaining soil fertility at a satisfactory level.

Depth of Soil

The importance of having an adequate depth of soil for storing sufficient amount of irrigation water and providing space for root penetration cannot be overemphasised. Shallow soils require more frequent irrigations and cause excessive deep percolation losses when shallow soils overlie coarse-textured and highly permeable sands and gravels. On the other hand, deep soils would generally require less frequent irrigations, permit the plant roots to penetrate deeper, and provide for large storage of irrigation water. As a result, actual water requirement for a given crop (or plant) is more in case of shallow soils than in deep soils even though the amount of water actually absorbed by the crop (or plant) may be the same in both types of soils. This is due to the unavoidable water losses at each irrigation.

CHEMICAL PROPERTIES OF SOIL

For satisfactory crop yield, soils must have sufficient plant nutrients, such as nitrogen, carbon, hydrogen, iron, oxygen, potassium, phosphorus, sulphur, magnesium, and so on. Nitrogen is the most important of all the nutrients. Nitrogenous matter is supplied to the soil from barnyard manure or from the growing of legume crops as green manures, or from commercial fertilisers. Plants absorb nitrogen in the form of soluble nitrates.

Soils having excess (greater than 0.15 to 0.20 per cent) soluble salts are called saline soils and those having excess of exchangeable sodium (more than 15 per cent or pH greater than 8.5) are called alkaline (or sodic) soils. Excessive amounts of useful plant nutrients such as sodium nitrate and potassium nitrate may become toxic to plants. Saline soils delay or prevent crop germination and also reduce the amount and rate of plant growth because of the high osmotic pressures which develop between the soil-water solution and the plants. These pressures adversely affect the ability of the plant to absorb water.

Alkaline (or sodic) soils tend to have inferior soil structure due to swelling of the soil particles. This changes the permeability of the soil. Bacterial environment is also an important feature of the soil-water-plant relationship. The formation of nitrates from nitrogenous compounds is accelerated due to favourable bacterial activity. Bacterial action also converts organic matter and other chemical compounds into forms usable by the plants. Bacterial activity is directly affected by the soil moisture, soil structure, and soil aeration. Compared to humid climate soils, arid soils provide better bacterial environment up to much greater depths because of their open structure. Besides, due to low rainfall in arid regions, leaching (i.e., draining away of useful salts) is relatively less and the arid soils are rich in mineral plant food nutrients, such as calcium and potassium.

Soils become saline or alkaline largely on account of the chemical composition of rocks weathering of which resulted in the formation of soils. Sufficient application of water to the soil surface through rains or irrigation helps in carrying away the salts from the root-zone region of the soil to the rivers and oceans. When proper drainage is not provided, the irrigation water containing excessive quantities of salt may, however, render the soil unsuitable for cultivation. Saline and alkaline soils can be reclaimed by: (i) adequate lowering of water table, (ii) leaching out excess salts, and (iii) proper management of soil so that the amount of salt carried away by the irrigation water is more than the amount brought in by irrigation water.

SOIL–WATER RELATIONSHIPS

Any given volume V of soil (Fig. 3.1) consists of : (i) volume of solids V_s , (ii) volume of liquids (water) V_w, and (iii) volume of gas (air) V_a. Obviously, the volume of voids (or pore spaces) $V_v = V_w + V_a$. For a fully saturated soil sample, $V_a = 0$ and $V_v = V_w$. Likewise, for a completely dry specimen, $V_w = 0$ and $V_v = V_a$. The weight of air is considered zero compared to the weights of water and soil grains. The void ratio e, the porosity n, the volumetric moisture content w, and the saturation S are defined as

$$e = \frac{V_v}{V_s},\quad n = \frac{V_v}{V},\quad w = \frac{V_w}{V},\quad S = \frac{V_w}{V_v}$$

Therefore, $w = Sn$ ...(3.1)

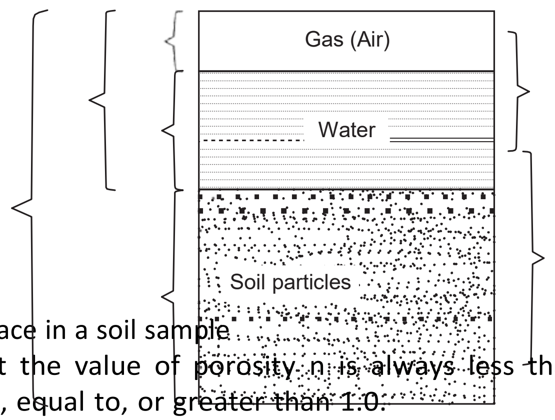


Fig. 3.1 Occupation of space in a soil sample

It should be noted that the value of porosity n is always less than 1.0. But, the value of void ratio e may be less, equal to, or greater than 1.0.

Further, if the weight of water in a wet soil sample is W_w and the dry weight of the sample is W_s , then the dry weight moisture fraction, W is expressed as (2)

$$W = \frac{W_w}{W_s} \qquad (3.2)$$

The bulk density (or the bulk specific weight or the bulk unit weight) γ_b of a soil mass is the total weight of the soil (including water) per unit bulk volume, i.e.,

$$\gamma_b = \frac{W_T}{V}$$

in which, $W_T = W_s + W_w$

The specific weight (or the unit weight) of the solid particles is the ratio of dry weight of the soil particles W_s to the volume of the soil particles V_s, i.e., W_s/V_s. Thus,

$$G_b \gamma_w = \frac{W_s}{V} \quad \text{i.e.,} \quad V = \frac{W_s}{G_b \gamma_w}$$

and

$$G_s \gamma_w = \frac{W_s}{V_s} \quad \text{i.e.,} \quad V_s = \frac{W_s}{G_s \gamma_w}$$

$$= \frac{W_s}{V} \qquad G_s \gamma_w$$

$$\therefore \quad \frac{V_s}{V} = \frac{G_b}{G_s} \qquad (3.3)$$

Here, γ_w is the unit weight of water and G_b and G_s are, respectively, the bulk specific gravity of soil and the relative density of soil grains. Further,

$$1 - n = 1 - \frac{V_v}{V} = \frac{V - V_v}{V} = \frac{V_s}{V} = \frac{G_b}{G_s}$$

$$\therefore \quad G_b = G_s(1 - n) \qquad (3.4)$$

Also, $$w = \frac{V_w}{V} = \frac{W_w / \gamma_w}{W_s / (G_b \gamma_w)} = G_b \frac{W_w}{W_s}$$

$$\therefore \quad w = G_b W \qquad (3.5)$$

and $$w = G_s(1 - n)W$$

Considering a soil of root-zone depth d and surface area A (i.e., bulk volume = Ad), $W_s = V_s G_s \gamma_w = Ad\,(1 - n)\,G_s \gamma_w$

Therefore, the dry weight moisture fraction, $W = \dfrac{W_w}{W_s}$

$$= \frac{V_w \gamma_w}{Ad\,(1 - n)\,G_s \gamma_w}$$

Therefore, the volume of water in the root-zone soil,

$$V_w = W\,Ad\,(1 - n)\,G_s \qquad (3.6)$$

This volume of water can also be expressed in terms of depth of water which would be obtained when this volume of water is spread over the soil surface area A.

$\therefore$ Depth of water, $d_w = \dfrac{V_w}{A}$

$$d_w = G_s\,(1 - n)\,Wd \qquad (3.7)$$

or $$d_w = w\,d \qquad (3.8)$$

Example 3.1 If the water content of a certain saturated soil sample is 22 per cent and the specific gravity is 2.65, determine the saturated unit weight γ_{sat}, dry unit weight γ_d, porosity n and void ratio e.

Solution:

$$W = \frac{W_w}{W_s} = 0.22$$

and $$G_s = \frac{W_s}{V_s \gamma_w} = 2.65$$

$$W_s = 2.65\,\gamma_w V_s$$

and $$WW_s = W_w$$

$$= 0.22 \times 2.65\,\gamma_w V_s$$

$V_w = \dfrac{W_w}{\gamma_w} = 0.22 \times 2.65 \times V_s = 0.583\ V_s$

Total volume $V = V_s + V_w$ (as $V_a = 0$ since the sample is saturated)

$= V_s (1 + 0.583)$

$= 1.583\ V_s$

$n = \dfrac{V_v}{V} = \dfrac{0.583\ V_s}{1.583\ V_s} = 36.8\%$ (since $V_v = V_w$ as the soil sample is saturated)

and $e = \dfrac{V_v}{V_s} = 0.583 = 58.3\%$

and total weight $W = W_w + W_s$

$= 0.22 \times 2.65 \times \gamma_w V_s + 2.65\ \gamma_w V_s$

$= 3.233\ \gamma_w V_s$

$\gamma_{sat} = \dfrac{W}{V} = \dfrac{3.233\ \gamma_w V_s}{1.583\ V_s}$

$= 20.032\ kN/m^3$ (since $\gamma_w = 9810\ N/m^3$)

$\gamma_d = \dfrac{W_s}{V} = \dfrac{2.65\ \gamma_w V_s}{1.583\ V_s}$

$= 16.422\ kN/m^3$.

Example 3.2 A moist clay sample weighs 0.55 N. Its volume is 35 cm^3. After drying in an oven for 24 hours, it weights 0.50 N. Assuming specific gravity of clay as 2.65, compute the porosity n, degree of saturation S, original moist unit weight, and dry unit weight.

Solution:

$W_T = 0.55$ N $W_s = 0.50$ N $W_w = 0.05$ N

$V_s = \dfrac{W_s}{\gamma_s} = \dfrac{0.5}{2.65 \times 9810}$

$= 1.923 \times 10^{-5}\ m^3 = 19.23\ cm^3$

$V_w = \dfrac{W_w}{\gamma_w} = \dfrac{0.05}{9810}$

$= 5.1 \times 10^{-6}\ m^3 = 5.10\ cm^3$

$V_v = V - V_s = 35 - 19.23$

$= 15.77\ cm^3$

Porosity, $n = \frac{V_v}{V} = \frac{15.77}{35} = 45.06\%$

Degree of saturation, $S = \frac{V_w}{V_v} = \frac{5.10}{15.77} = 32.34\%$

Moist unit weight, $\gamma = \frac{0.55}{35} = 0.016$ N/m^3

Dry unit weight, $\gamma_d = \frac{0.50}{35} = 0.014$ N/m^3.

Example 3.3 A moist soil sample has a volume of 484 cm^3 in the natural state and a weight of 7.94N. The dry weight of the soil is 7.36 N and the relative density of the soil particles is 2.65. Determine the porosity, soil moisture content, volumetric moisture content, and degree of saturation.

Solution:

$$G_b = \frac{7.36}{484 \times 10^{-6} \times 9810} = 1.55$$

The porosity, $n = 1 - \frac{G_b}{G_s}$

$= 1 - \frac{1.55}{2.65} = 0.415 = 41.5\%$

The soil moisture fraction,

$W = \frac{7.94 - 7.36}{7.36} = 0.0788 = 7.88\%$

The volumetric moisture content,

G_b W = 1.55 (0.0788)

= 12.214%

Degree of saturation, $S = \frac{w}{n} = \frac{12.214}{41.5} = 0.2943 = 29.43\%$

ROOT-ZONE SOIL WATER

Water serves the following useful functions in the process of plant growth:

Germination of seeds,

All chemical reactions,

All biological processes,

Absorption of plant nutrients through their aqueous solution,

Temperature control,

Tillage operations, and

Washing out or dilution of salts.

Crop growth (or yield) is directly affected by the soil moisture content in the root zone. The root zone is defined as the volume of soil or fractured rock occupied or occupiable by roots of the plants from which plants can extract water (3). Both excessive water (which results in waterlogging) and deficient water in the root-zone soil retard crop growth and reduce the crop yield.

Soil water can be divided into three categories:

Gravity (or gravitational or free) water,

Capillary water, and

Hygroscopic water.

Gravity water is that water which drains away under the influence of gravity. Soon after irrigation (or rainfall) this water remains in the soil and saturates the soil, thus preventing circulation of air in void spaces.

The capillary water is held within soil pores due to the surface tension forces (against gravity) which act at the liquid-vapour (or water-air) interface.

Water attached to soil particles through loose chemical bonds is termed hygroscopic water. This water can be removed by heat only. But, the plant roots can use a very smallfraction of this moisture under drought conditions.

When an oven-dry (heated to 105°C for zero per cent moisture content) soil sample is exposed to atmosphere, it takes up some moisture called hygroscopic moisture. If more water is made available, it can be retained as capillary moisture due to surface tension (i.e., intermolecular forces). Any water, in excess of maximum capillary moisture, flows down freely and is the gravitational (or gravity) water.

The water remaining in the soil after the removal of gravitational water is called the field capacity. Field capacity of a soil is defined as the moisture content of a deep, permeable, and well-drained soil several days after a thorough wetting. Field capacity is measured in terms of the moisture fraction, $W_{fc} = (W_w/W_s)$ of the soil when, after thorough wetting of the soil, free drainage (at rapid rate) has essentially stopped and further drainage, if any, occurs at a very slow rate. An irrigated soil, i.e., adequately wetted soil, may take approximately one (in case of sandy soil) to three (in case of clayey soil) days for the rapid drainage to stop. This condition corresponds to a surface tension of one-tenth bar (in case of sandy soils) to one-third bar (in case of clayey soils). Obviously, the field capacity depends on porosity and soil moisture tension. The volumetric moisture content at the field capacity w_{fc} becomes equal to $G_b\ W_{fc}$.

Plants are capable of extracting water from their root-zone soil to meet their transpiration demands. But, absence of further addition to the soil moisture may result in very low availabilityof soil water and under such a condition the water is held so tightly in the soil pores that the rate of water absorption by plants may not meet their transpiration demands and the plantsmay either wilt or even die, if not supplied with water immediately and well before the plantswilt. After wilting, however, a plant may not regain its strength and freshness even if the soil is saturated with water. Permanent wilting point is defined as the soil moisture fraction, W_{wp} at which the plant leaves wilt (or droop) permanently and applying additional water after this stage will not relieve the wilted condition. The soil moisture tension at this condition is around 15 bars (2). The moisture content at the permanent wilting condition will be higher in a hot climate than in a cold climate. Similarly, the percentage of soil moisture at the permanent wilting point of a plant will be larger in clayey soil than in sandy soil. The permanent wilting point is, obviously, at the lower end of the available moisture range and can be approximately estimated by dividing the field capacity by a factor varying from 2.0 (for soils with low silt content) to 2.4 (for soils with high silt content). The permanent wilting point also depends upon the nature of crop. The volumetric moisture content at the permanent wilting point, w_{wp} becomes $G_b W_{wp}$. Figure 3.2 shows different stages of soil moisture content in a soil and the corresponding conditions.

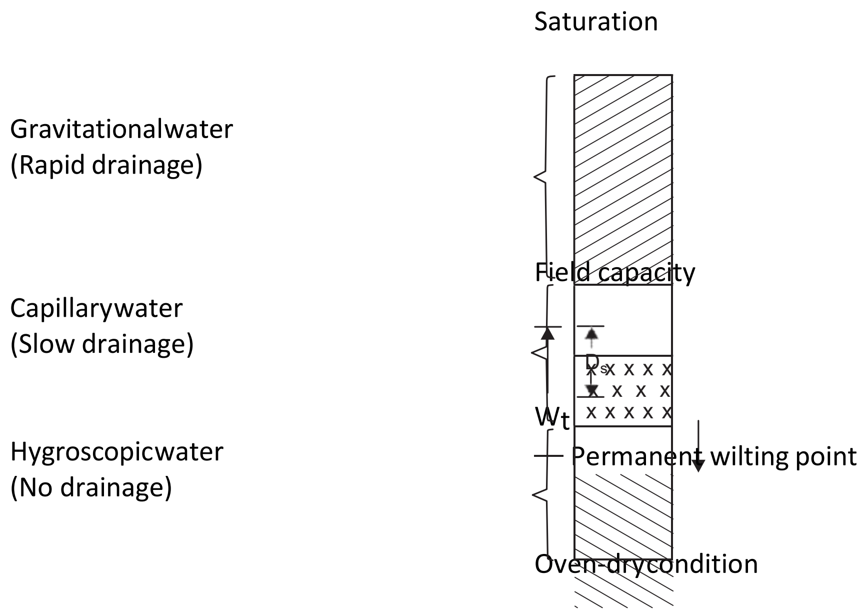

Saturation
Gravitationalwater
(Rapid drainage)
Field capacity
Capillarywater
(Slow drainage)
D
Wt
Hygroscopicwater
(No drainage)
Permanent wilting point
Oven-drycondition

Fig. 3.2 Different stages of soil moisture content in a soil

The difference in the moisture content of the soil between its field capacity and the permanent wilting point within the root zone of the plants is termed available moisture. It represents the maximum moisture which can be stored in the soil for plant use. It should be noted that the soil moisture content near the wilting point is not easily extractable by the plants. Hence, the term readily available moisture is used to represent that fraction of the available moisture which can be easily extracted by the plants. Readily available moisture is approximately 75% of the available moisture.

The total available moisture d_t (in terms of depth) for a plant (or soil) is given by

$$d_t = (w_{fc} - w_{wp})\, d \qquad (3.9)$$

in which, d is the depth of the root zone.

It is obvious that soil moisture can vary between the field capacity (excess amount would drain away) and the permanent wilting point. However, depending upon the prevailing conditions, soil moisture can be allowed to be depleted below the field capacity (but not below the permanent wilting point in any case), before the next irrigation is applied. The permissible amount of depletion is referred to as the management allowed deficit D_m which primarily depends on the type of crop and its stage of growth (2). Thus,

$$D_m = f_m\, d_t \qquad (3.10)$$

in which, f_m is, obviously, less than 1 and depends upon the crop and its stage of growth. At a time when the soil moisture content is w, the soil-moisture deficit D_s is given as

$$D_s = (w_{fc} - w)\, d \qquad (3.11)$$

Example 3.5 The field capacity and permanent wilting point for a given 0.8 m root-zone soil are 35 and 10 per cent, respectively. At a given time, the soil moisture in the given soil is 20 per cent when a farmer irrigates the soil with 250 mm depth of water. Assuming bulk specific gravity of the soil as 1.6, determine the amount of water wasted from the consideration of irrigation.

Solution:

At the time of application of water,

Soil moisture deficit, $D_s = (W_{fc} - W)\, d\, G_b$

$= (0.35 - 0.20)\,(0.8)\,(1.6)$

$= 0.192$ m

Therefore, the amount of water wasted

$= 0.250 - 0.192$

$= 58$ mm

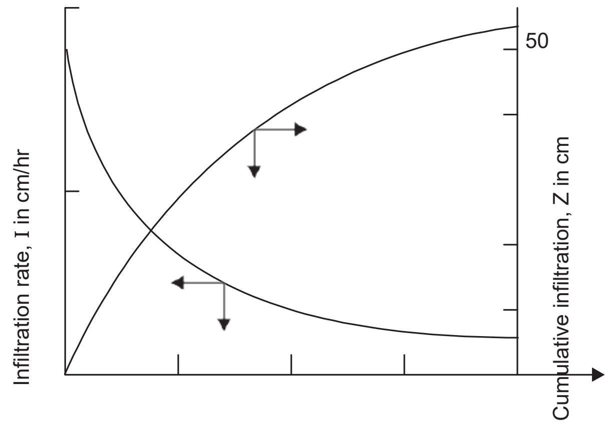
$$= \frac{58}{250} \times 100 = 23.2\%$$

INFILTRATION

Infiltration is another important property of soil which affects surface irrigation. It not only controls the amount of water entering the soil but also the overland flow. Infiltration is a complex process which depends on: (i) soil properties, (ii) initial soil moisture content, (iii) previous wetting history, (iv) permeability and its changes due to surface water movement, (v) cultivation practices, (vi) type of crop being sown, and (vii) climatic effects. In an initially dry soil, the infiltration rate is high at the beginning of rain (or irrigation), but rapidly decreases with time until a fairly steady state infiltration is reached (Fig. 3.3). This constant rate of infiltration is also termed the basic infiltration rate and is approximately equal to the permeability of the saturated soil.

The moisture profile under ponded infiltration into dry soil, Fig. 3.4, can be divided into the following five zones (4):

10



0

0 Infiltration time, hours 20

Fig. 3.3 Variation of infiltration rate, I and cumulative infiltration, Z with time

O θ_i Water content θ_s

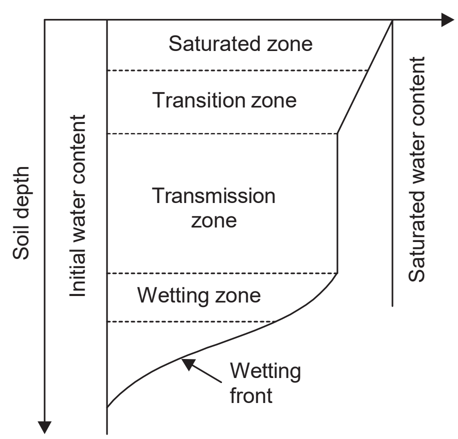


Fig. 3.4 Soil-moisture profile during ponded infiltration

The saturated zone extending up to about 1.5 cm below the surface and having a saturated water content.

The transition zone which is about 5 cm thick and is located below the saturated zone. In this zone, a rapid decrease in water content occurs.

The transmission zone in which the water content varies slowly with depth as well as time.

The wetting zone in which sharp decrease in water content is observed.

The wetting front is a region of very steep moisture gradient. This represents the limit of moisture penetration into the soil.

Table 3.1 lists the ranges of porosity, field capacity, permanent wilting point, and basic infiltration rate (or permeability) for different soil textures.

Table 3.1 Representative properties of soil

Soil texture	Porosity (%)	Field capacity (%)	Permanent wilting point (%)	Basic infiltration rate (cm/hr)
Sand	32-42	5-10	2-6	2.5-25
Sandy loam	40-47	10-18	4-10	1.3-7.6
Loam	43-49	18-25	8-14	0.8-2.0
Clay loam	47-51	24-32	11-16	0.25-1.5
Silty clay	49-53	27-35	13-17	0.03-0.5
Clay	51-55	32-40	15-22	0.01-0.1

CONSUMPTIVE USE (OR EVAPOTRANSPIRATION)

The combined loss of water from soil and crop by vaporisation is identified as evapotranspiration (3). Crops need water for transpiration and evaporation. During the growing period of a crop, there is a continuous movement of water from soil into the roots, up the stems and leaves, and out of the leaves to the atmosphere. This movement of water is essential for carrying plant food from the soil to various parts of the plant. Only a very small portion (less than 2 per cent) of water absorbed by the roots is retained in the plant and the rest of the absorbed water, after performing its tasks, gets evaporated to the atmosphere mainly through the leaves and stem. This process is called transpiration. In addition, some water gets evaporated to the atmosphere directly from the adjacent soil and water surfaces and from the surfaces of the plant leaves (i.e., the intercepted precipitation on the plant foliage). The water needs of a crop thus consists of transpiration and evaporation and is called evapotranspiration or consumptive use.

Consumptive use refers to the water needs of a crop in a specified time and is the sum ofthe volume of transpirated and evaporated water. Consumptive use is defined as the amount of water needed to meet the water loss through evapotranspiration. It generally applies to a crop but can be extended to a field, farm, project or even a valley. Consumptive use is generally measured as volume per unit area or simply as the depth of water on the irrigated area. Knowledge of consumptive use helps determine irrigation requirement at the farm which should, obviously, be the difference between the consumptive use and the effective precipitation.

Evapotranspiration is dependent on climatic conditions like temperature, daylight hours, humidity, wind movement, type of crop, stage of growth of crop, soil moisture depletion, andother physical and chemical properties of soil. For example, in a sunny and hot climate, crops need more water per day than in a cloudy and cool climate. Similarly, crops like rice or sugarcane need more water than crops like beans and wheat. Also, fully grown crops need more water than crops which have been just planted.

While measuring or calculating potential evapotranspiration, it is implicitly assumed that water is freely available for evaporation at the surface. Actual evapotranspiration, in the absence of free availability of water for evaporation will, obviously, be less and is determined by: (i) the extent to which crop covers the soil surface, (ii) the stage of crop growth which affects the transpiration and soil surface coverage, and (iii) soil water supply.

Potential evapotranspiration is measured by growing crops in large containers, known as lysimeters, and measuring their water loss and gains. Natural conditions are simulated in these containers as closely as possible. The operator measures water added, water retained by the soil, and water lost through evapotranspiration and deep percolation. Weighings can be made with scales or by floating the lysimeters in water. Growth of roots in lysimeters confined to the dimensions of lysimeters, the disturbed soil in the lysimeters and other departures from natural conditions limit the accuracy of lysimeter measurements of potential evapotranspiration. Potential evapotranspiration from a cropped surface can be estimated either by correlating potential evapotranspiration with water loss from evaporation devices or by estimations based on various climatic parameters. Correlation of potential evapotranspiration

assumes that the climatic conditions affecting crop water loss (D_{et}) and evaporation from a free surface of water (E_p) are the same. Potential evapotranspiration D_{et} can be correlated to the pan evaporation E_p as (3),

$D_{et} = KE_p$ (3.12)

in which, K is the crop factor for that period. Pan evaporation data for various parts of India

are published by the Meteorological Department. The crop factor K depends on the crop as well as its stage of growth (Table 3.2). The main limitations of this method are the differences in physical features of evaporation surfaces compared with those of a crop surface.

Table 3.2 Values of crop factor K from some major crops

Percentage of crop growing season since sowing	Maize, cotton, potatoes, peas and sugarbeets	Wheat, barley and other small grains	Sugarcane	Rice
0	0.20	0.08	0.50	0.80
10	0.36	0.15	0.60	0.95
25	0.75	0.33	0.75	1.10
50	1.00	0.65	1.00	1.30
75	0.85	0.90	0.85	1.15
100	0.20	0.20	0.50	0.20

In the absence of pan evaporation data, the consumptive use is generally computed as follows:

Compute the seasonal (or monthly) distribution of potential evapotranspiration, which is defined as the evapotranspiration rate of a well-watered reference crop which com- pletely shades the soil surface (2). It is thus an indication of the climatic evaporation demand of a vigorously growing crop. Usually, grass and alfalfa (a plant with leaves like that of clover and purple flowers used as food for horses and cattle) are taken asreference crops.

Adjust the potential evapotranspiration for the type of crop and the stage of crop growth. Factors such as soil moisture depletion are ignored so that the estimated values of the consumptive use are conservative values to be used for design purposes.

Thus, evapotranspiration of a crop can be estimated by multiplying potential evapotranspiration by a factor known as crop coefficient.

Potential evapotranspiration can be computed by one of the several methods available for the purpose. These methods range in sophistication from simple temperature correlation (such as the Blaney-Criddle formula) to equations (such as Penman's equation) which account for radiation energy as well. Blaney-Criddle formula for the consumptive use has been usedextensively and is expressed as (1)

$$u = kf \quad (3.13)$$

in which, u = consumptive use of crop in mm,

k = empirical crop consumptive use coefficient (Table 3.3), and

f = consumptive use factor.

The quantities u, k, and f are determined for the same period (annual, irrigation season, growing season or monthly). The consumptive use factor f is expressed as

$$f = \frac{p}{100}(1.8t + 32) \quad (3.14)$$

in which, t = mean temperature in °C for the chosen period, and

p = percentage of daylight hours of the year occurring during the period.

Table 3.4 lists the values of p for different months of a year for 0° north latitude. The value of the consumptive use is generally determined on a monthly basis and the irrigation system must be designed for the maximum monthly water needs. It should be noted that Eq. (3.13) was originally in FPS system with appropriate values of k. Similarly, Eq. (3.14) too had a different form with t in Fahrenheit.

Table 3.3 Consumptive use coefficient for some major crops (1)

Crop	Lenght of normal growing season orperiod		Consumptive use coefficient, k			
			For the growing period*		Monthly (maximumvalue)**	
Corn (maize)	4	months	19.05 to	21.59	20.32 to	30.48
Cotton	7	months	15.24 to	17.78	19.05 to	27.94
Potatoes	3-5	months	16.51 to	19.05	21.59 to	25.40
Rice	3-5	months	25.40 to	27.94	27.94 to	33.02
Small grains	3	months	19.05 to	21.59	21.59 to	25.40
Sugarbeet	6	months	16.51 to	19.05	21.59 to	25.40
Sorghums	4-5	months	17.78 to	20.32	21.59 to	25.40
Orange and lemon	1	year	11.43 to	13.97	16.21 to	19.05

*The lower values are for more humid areas and the higher values are for more arid climates.

** Dependent upon mean monthly temperature and stage of growth of crop.

Table 3.4 Per cent daylight hours for northern hemispere (0-50° latitude) (1)

Latitude North (in degr-ees)	Jan.	Fab.	March	April	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.
0	8.50	7.66	8.49	8.21	8.50	8.22	8.50	8.49	8.21	8.50	8.22	8.50
5	8.32	7.57	8.47	8.29	8.65	8.41	8.67	8.60	8.23	8.42	8.07	8.30

10	8.13	7.47	8.45	8.37	8.81	8.60	8.86	8.71	8.25	8.34	7.91	8.10
15	7.94	7.36	8.43	8.44	8.98	8.80	9.05	8.83	8.28	8.26	7.75	7.88

(Contd.)...

20	7.74	7.25	8.41	8.52	9.15	9.00	9.25	8.96	8.30	8.18	7.58	7.66
25	7.53	7.14	8.39	8.61	9.33	9.23	9.45	9.09	8.32	8.09	7.40	7.42
30	7.30	7.03	8.38	8.72	9.53	9.49	9.67	9.22	8.33	7.99	7.19	7.15
32	7.20	6.97	8.37	8.76	9.62	9.59	9.77	9.27	8.34	7.95	7.11	7.05
34	7.10	6.91	8.36	8.80	9.72	9.70	9.88	9.33	8.36	7.90	7.02	6.92
36	6.99	6.85	8.35	8.85	9.82	9.82	9.99	9.40	8.37	7.85	6.92	6.79
38	6.87	6.79	8.34	8.90	9.92	9.95	10.10	9.47	8.38	7.80	6.82	6.66
40	6.76	6.72	8.33	8.95	10.02	10.08	10.22	9.54	8.39	7.75	6.72	6.52
42	6.63	6.65	8.31	9.00	10.14	10.22	10.35	9.62	8.40	7.69	6.62	6.37
44	6.49	6.58	8.30	9.06	10.26	10.38	10.49	9.70	8.41	7.63	6.49	6.21
46	6.34	6.50	8.29	9.12	10.39	10.54	10.64	9.79	8.42	7.57	6.36	6.04
48	6.17	6.41	8.27	9.18	10.53	10.71	10.80	9.89	8.44	7.51	6.23	5.86
50	5.98	6.30	8.24	9.24	10.68	10.91	10.99	10.00	8.46	7.45	6.10	5.65

Table 3.5 gives typical values of the water needs of some major crops for the total growing period of some of the crops (5). This table also indicates the sensitivity of the crop to water shortages or drought. High sensitivity to drought means that the crop cannot withstand watershortages, and that such shortages should be avoided.

Table 3.5 Indicative values of crop water needs and sensitivity to drought (5)

Crop	Crop water need (mm/total growing period)	Sensitivity of Drought
Alfalfa	800 - 1600	low - Medium
Banana	1200 - 2200	high
Barley/oats/wheat	450 - 650	low - Medium
Bean	300 - 500	medium - High
Cabbage	350 - 500	medium - High
Citrus	900 - 1200	low - Medium
Cotton	700 - 1300	low
Maize	500 - 800	medium - High
Melon	400 - 600	medium - High
Onion	350 - 550	medium - High
Peanut	500 - 700	low - Medium
Pea	350 - 500	medium - High
Pepper	600 - 900	medium - High
Potato	500 - 700	high
Rice (paddy)	450 - 700	high
Sorghum/millet	450 - 650	low
Soybean	450 - 700	low - Medium
Sugarbeet	550 - 750	low - Medium
Sugarcane	1500 - 2500	high
Sunflower	600 - 1000	low - Medium
Tomato	400 - 800	medium - High

Example 3.6 Using the Blaney-Criddle formula, estimate the yearly consumptive use of water for sugarcane for the data given in the first four columns of Table 3.6.

Solution:

According to Eqs. (3.13) and (3.14),

$$u = k \frac{p}{100} (1.8\,t + 32)$$

Values of monthly consumptive use calculated from the above formula have been tabulated in the last column of Table 3.6. Thus, yearly consumptive use = $\sum u$ = 1.75 m.

Table 3.6 Data and solution for Example 3.6

Month	Mean monthly temperature, t°C	Monthly crop coefficient, k	Per cent sunshine hours, p	Monthly consumptive use, u (mm)
January	13.10	19.05	7.38	78.14
February	15.70	20.32	7.02	85.96
March	20.70	21.59	8.39	125.46
April	27.00	21.59	8.69	151.22
May	31.10	22.86	9.48	190.66
June	33.50	24.13	9.41	209.58
July	30.60	25.40	9.60	212.34
August	29.00	25.40	9.60	205.31
September	28.20	24.13	8.33	166.35
October	24.70	22.86	8.01	140.01
November	18.80	21.59	7.25	103.06
December	13.70	19.05	7.24	78.15

IRRIGATION REQUIREMENT

Based on the consumptive use, the growth of all plants can be divided into three stages, viz., vegetative, flowering, and fruiting. The consumptive use continuously increases during the vegetative stage and attains the peak value around the flowering stage; thereafter, the consumptive use decreases. It should be noted that different crops are harvested during different stages of crop growth. For example, leafy vegetables are harvested during the vegetative stage and flowers are harvested during the flowering stage. Most crops (such as potatoes, rice, corn, beans, bananas, etc.) are harvested during the fruiting stage.

At each precipitation, a certain volume of water is added to the crop field. Not all of the rainfall can be stored within the root zone of the soil. The part of the precipitation which has gone as surface runoff, percolated deep into the ground or evaporated back to the atomosphere does not contribute to the available soil moisture for the growth of crop. Thus, effective precipitation is only that part of the precipitation which contributes to the soil moisture available for plants. In other words, the effective rainfall is the water retained in the root zone and is obtained by subtracting the sum of runoff, evaporation, and deep percolation from the total rainfall.

If, for a given period, the consumptive use exceeds the effective precipitation, the difference has to be met by irrigation water. In some cases irrigation water has to satisfy leaching requirements too. Further, some of the water applied to the field necessarily flows away as surface runoff and/or percolates deep into the ground and/or evaporates to the atmosphere. Therefore, irrigation requirement is the quantity of water, exclusive of precipitation and regardless of its source, required by a crop or diversified pattern of crops in a given period of time of their normal growth under field conditions. It includes evapotranspiration not met by effective precipitation and other economically unavoidable losses such as surface runoff and deep percolation. Irregular land surfaces, compact impervious soils or shallow soils over a gravel stratum of high permeability, small or too large irrigation streams, absence of an attendant during irrigation, long irrigation runs, improper land preparation, steep ground slopes and such other factors contribute to large losses of irrigation water which, in turn, reduce irrigation efficiency. Irrigation efficiency is the ratio of irrigation water consumed by crops of an irrigated field to the water diverted from the source of supply. Irrigation efficiency is usually measured at the field entrance (3). Water application efficiency is the ratio of the average depth added to the root-zone storage to the average depth applied to the field. Obviously, irrigation efficiency measured at the field and the water application efficiency would be the same. Thus, the field irrigation requirement FIR is expressed as (2)

$$FIR = \frac{D_{et} - (D_p - D_{pl})}{E_a} \qquad (3.15)$$

in which, D_{et} = depth of evapotranspiration,

D_p = depth of precipitation,

D_{pl} = depth of precipitation that goes as surface runoff and/or infiltrates into the ground and/or intercepted by the plants,

and E_a = irrigation efficiency or application efficiency.

In the absence of any other information, the following values can be used as a guide for E_a in different methods of surface irrigation for different types of soils:

Soil class	Irrigation efficiency (%)
Sand	60
Sandy loam	65
Loam	70
Clay loam	75
Heavy clay	80

If no other information is available, the following formulae can be used to estimate the effective rainfall depth, D_{pe} provided that the ground slope does not exceed 5%.

$D_{pe} = 0.8\ D_p - 25$ if $D_p > 75$ mm/month

$D_{pe} = 0.6\ D_p - 10$ if $D_p < 75$ mm/month

D_{pe} is always equal to or greater than zero and never negative. Both D_p ad D_{pe} are in mm/month in the foregoing formulae.

Example 3.7 Using the data given in the first four columns of Table 3.7 for a given crop, determine the field irrigation requirement for each month assuming irrigation efficiency to be 60 per cent.

Table 3.7 Data and solution for Example 3.5

Month	Crop factor, K	Pan evaporation, E_p (mm)	Effective rain-fall, $D_p - D_{pl}$ (mm)	Consumptive use, D_{et} (mm)	FIR (mm)
November	0.20	118.0	6.0	23.60	29.33
December	0.36	96.0	16.0	34.56	30.93
January	0.75	90.0	20.0	67.50	79.17
February	0.90	105.0	15.0	94.50	132.50
March	0.80	140.0	2.0	112.00	183.33

Solution:

According to Eqs. (3.12) and (3.15)

$D_{et} = KE_p$

and $\quad FIR = \dfrac{D_{et} - (D_p - D_{pl})}{E_a}$

Given $E_a = 0.6$

Field irrigation requirement calculated for each month of the crop-growing season has been tabulated in the last column of Table 3.7.

FREQUENCY OF IRRIGATION

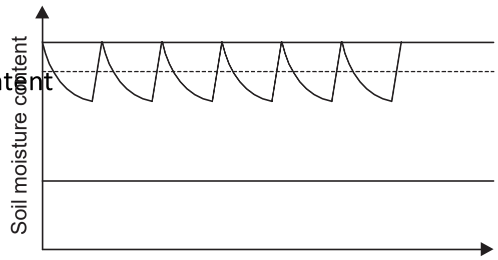
Growing crops consume water continuously. However, the rate of consumption depends on the type of crop, its age, and the atmospheric conditions all of which are variable factors. The aim of each irrigation is to fulfil the needs of the crop for a period which may vary from few days to several weeks. The frequency of irrigation primarily depends on: (i) the water needs of the crop, (ii) the availability of water, and (iii) the capacity of the root-zone soil to store water. Shallow-rooted crops generally require more frequent irrigation than deep-rooted crops. The roots of a plant in moist soil extract more water than the roots of the same plant in drier soil.

A moderate quantity of soil moisture is beneficial for good crop growth. Both excessive and deficient amount of soil moisture retard the crop growth and thus the yield. Excessive flooding drives out air which is essential for satisfactory crop growth. In case of deficient moisture, the plant has to spend extra energy to extract the desired amount of water.

Many of the crops have an optimum soil moisture content at which the yield is maximum; if the moisture content is less or more than this amount, the yield reduces. Wheat has a well-defined optimum moisture content of around 40 cm. However, there are other crops in which the yield initially increases at a much faster rate with the increase in the soil moisture content and the rate of increase of the yield becomes very small at higher moisture content. In such cases, the soil moisture is kept up to a level beyond which the increase in production is not worth the cost of the additional water supplied.

It should be noted that, because of the capacity of a soil to store water, it is not necessary to apply water to the soil every day even though the consumptive use takes place continuously. The soil moisture can vary between the field capacity and the permanent wilting point. The average moisture content will thus depend on the frequency of irrigation and quantity of water applied. As can be seen from Fig. 3.5, frequent irrigation (even of smaller depths) keeps the average mositure content closer to the field capacity. On the other hand, less frequent irrigation of larger depths of water will keep the average moisture content on the lower side.

Field capacity
Average moisture content



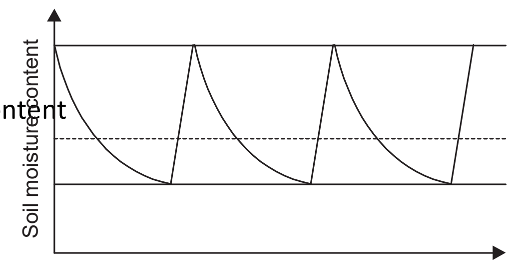
PWP

Time

MORE FREQUENT IRRIGATION

Field capacity

Average moisture content
PWP



Time

LESS FREQUENT IRRIGATION

Fig. 3.5 Effect of frequency of irrigation on average moisute content

For most of the crops, the yield remains maximum if not more than 50 per cent of the available water is removed during the vegetative, flowering, and the intitial periods of the fruiting stage. During the final period of the fruiting stage, 75 per cent of the available moisture can be depleted without any adverse effect on the crop yield.

The frequency of irrigation (or irrigation interval) is so decided that the average moisture content is close to the optimum and at each irrigation the soil moisture content is brought to the field capacity. Alternatively, the frequency of irrigation can be decided so as to satisfy the daily consumptive use requirement which varies with stage of growth. Thus, frequency of irrigation is calculated by dividing the amount of soil moisture which may be depleted (i.e., allowable depletion below field capacity and well above permanent wilting point) within the root-zone soil by the rate of consumptive use. Thus,

$$\text{Frequency of irrigation} = \frac{\text{Allowable soil moisture depletion}}{\text{Rate of consumptive use}} \tag{3.16}$$

The depth of watering at each irrigation to bring the moisture content w to the field capacity w_{fc} in a soil of depth d can be determined from the following relation:

$$\text{Depth of water to be applied} = \frac{(w_{fc} - w)d}{E_a} \tag{3.17}$$

Example 3.8 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%, and

0% of the maximum depth of available water in the root zone which is 80 mm. Assume irrigation efficiency to be 65%.

Solution:

Frequency of irrigation $= \dfrac{80 \times (1 - 0.25)}{2.8}$

= 21.43 days

= 21 days (say)

(ii) Depth of water to be applied $= \dfrac{80 \times (1 - 0.25)}{0.65}$

= 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

METHODS OF IRRIGATION

Any irrigation system would consist of the following four subsystems (2):

The water supply subsystem which may include diversion from rivers or surface ponds or pumped flow of ground water.

The water delivery subsystem which will include canals, branches, and hydraulic structures on these.

The water use subsystems, which can be one of the four main types, namely, (a) surface irrigation, (b) subsurface irrigation, (c) sprinkler irrigation, and (d) trickle irrigation.

The water removal system i.e., the drainage system.

In this section, the water use subsystems have been described.

Water Use Subsystems

Irrigation water can be applied to croplands using one of the following irrigation methods (1):

Surface irrigation which includes the following:

Uncontrolled (or wild or free) flooding method,

Border strip method,

Check method,

Basin method, and

Furrow method.

Subsurface irrigation

Sprinkler irrigation

Trickle irrigation

Each of the above methods has some advantages and disadvantages, and the choice ofthe method depends on the following factors (2):

Size, shape, and slope of the field,

Soil characteristics,

Nature and availability of the water supply subsystem,

Types of crops being grown,

Initial development costs and availability of funds, and

Preferences and past experience of the farmer.

The design of an irrigation system for applying water to croplands is quite complex andnot amenable to quantitative analysis. Principal criteria for the design of a suitable irrigation method are as follows (3):

Store the required water in the root-zone of the soil,

Obtain reasonably uniform application of water,

Minimise soil erosion,

Minimise run-off of irrigation water from the field,

Provide for beneficial use of the runoff water,

Minimise labour requirement for irrigation,

Minimise land use for ditches and other controls to distribute water,

Fit irrigation system to field boundaries,

Adopt the system to soil and topographic changes, and

Facilitate use of machinery for land preparation, cultivating, furrowing, harvesting,and so on.

Surface Irrigation

In all the surface methods of irrigation, water is either ponded on the soil or allowed to flow continuously over the soil surface for the duration of irrigation. Although surface irrigation is the oldest and most common method of irrigation, it does not result in high levels of performance. This is mainly because of uncertain infiltration rates which are affected by year-to-year changes in the cropping pattern, cultivation practices, climatic factors, and many other factors. As a result, correct estimation of irrigation efficiency of surface irrigation is difficult. Application efficiencies for surface methods may range from about 40 to 80 per cent.

Uncontrolled Flooding

When water is applied to the cropland without any preparation of land and without any levees to guide or restrict the flow of water on the field, the method is called ‘uncontrolled’, wild or ‘free’ flooding. In this method of flooding, water is brought to field ditches and then admitted at one end of the field thus letting it flood the entire field without any control.

Uncontrolled flooding generally results in excess irrigation at the inlet region of thefield and insufficient irrigation at the outlet end. Application efficiency is reduced because of either deep percolation (in case of longer duration of flooding) or flowing away of water (in caseof shorter flooding duration) from the field. The application efficiency would also depend on the depth of flooding, the rate of intake of water into the soil, the size of the stream, and topography of the field.

Obviously, this method is suitable when water is available in large quantities, the land surface is irregular, and the crop being grown is unaffected because of excess water. The advantage of this method is the low initial cost of land preparation. This is offset by the disadvantage of greater loss of water due to deep percolation and surface runoff.

Border Strip Method

Border strip irrigation (or simply 'border irrigation') is a controlled surface flooding method of applying irrigation water. In this method, the farm is divided into a number of strips which can be 3-20 metres wide and 100-400 metres long. These strips are separated by low levees (or borders). The strips are level between levees but slope along the length according to natural slope. If possible, the slope should be between 0.2 and 0.4 per cent. But, slopes as flat as 0.1 per cent and as steep as 8 per cent can also be used (1). In case of steep slope, care should be taken to prevent erosion of soil. Clay loam and clayey soils require much flatter slopes (around 0.2%) of the border strips because of low infiltration rate. Medium soils may have slopes ranging from 0.2 to 0.4%. Sandy soils can have slopes ranging from 0.25 to 0.6%.

Water from the supply ditch is diverted to these strips along which it flows slowly towards the downstream end and in the process it wets and irrigates the soil. When the water supply is stopped, it recedes from the upstream end to the downsteam end.

The border strip method is suited to soils of moderately low to moderately high intake rates and low erodibility. This method is suitable for all types of crops except those which require prolonged flooding which, in this case, is difficult to maintain because of the slope. This method, however, requires preparation of land involving high initial cost.

Check Method

The check method of irrigation is based on rapid application of irrigation water to a level or nearly level area completely enclosed by dikes. In this method, the entire field is divided into a number of almost levelled plots (compartments or 'Kiaries') surrounded by levees. Water is admitted from the farmer's watercourse to these plots turn by turn. This method is suitable for a wide range of soils ranging from very permeable to heavy soils. The farmer has very good control over the distribution of water in different areas of his farm. Loss of water through deep percolation (near the supply ditch) and surface runoff can be minimised and adequate irrigation of the entire farm can be achieved. Thus, application efficiency is higher for this method. However, this method requires constant attendance and work (allowing and closing the supplies to the levelled plots). Besides, there is some loss of cultivable area which is occupied by the levees. Sometimes, levees are made sufficiently wide so that some 'row' crops can be grown over the levee surface.

Basin Method

This method is frequently used to irrigate orchards. Generally, one basin is made for one tree. However, where conditions are favourable, two or more trees can be included in one basin.

Furrow Method

In the surface irrigation methods discussed above, the entire land surface is flooded during each irrigation. An alternative to flooding the entire land surface is to construct small channels along the primary direction of the movement of water and letting the water flow through these channels which are termed 'furrows', 'creases' or 'corrugation'. Furrows are small channels having a continuous and almost uniform slope in the direction of irrigation. Water infiltrates through the wetted perimeter of the furrows and moves vertically and then laterally to saturate the soil. Furrows are used to irrigate crops planted in rows.

Furrow lengths may vary from 10 metres to as much as 500 metres, although, 100 metres to 200 metres are the desirable lengths and more common. Very long furrows may result in excessive deep percolation losses and soil erosion near the upstream end of the field. Preferable slope for furrows ranges between 0.5 and 3.0 per cent. Many different classes of soil have been satisfactorily irrigated with furrow slope ranging from 3 to 6 per cent (1). In case of steep slopes, care should be taken to control erosion. Spacing of furrows for row crops (such as corn, potatoes, sugarbeet, etc.) is decided by the required spacing of the plant rows. The furrow stream should be small enough to prevent the flowing water from coming in direct contact with the plant. Furrows of depth 20 to 30 cm are satisfactory for soils of low permeability. Forother soils, furrows may be kept 8 to 12 cm deep.

Water is distributed to furrows from earthen ditches through small openings made in earthen banks. Alternatively, a small-diameter pipe of light weight plastic or rubber can be used to siphon water from the ditch to the furrows without disturbing the banks of the earthen ditch.

Furrows necessitate the wetting of only about half to one-fifth of the field surface. This reduces the evaporation loss considerably. Besides, puddling of heavy soils is also lessened and it is possible to start cultivation soon after irrigation. Furrows provide better on-farm water management capabilities for most of the surface irrigation conditions, and variable and severe topographical conditions. For example, with the change in supply conditions, number of simultaneously supplied furrows can be easily changed. In this manner, very high irrigationefficiency can be achieved.

The following are the disadvantages of furrow irrigation:

Possibility of increased salinity between furrows,

Loss of water at the downstream end unless end dikes are used,

The necessity of one extra tillage work, viz., furrow construction,

Possibility of increased erosion, and

Furrow irrigation requires more labour than any other surface irrigation method.

Subsurface Irrigation

Subsurface irrigation (or simply subirrigation) is the practice of applying water to soils directly under the surface. Moisture reaches the plant roots through capillary action. The conditions which favour subirrigation are as follows (1):

Impervious subsoil at a depth of 2 metres or more,

A very permeable subsoil,

A permeable loam or sandy loam surface soil,

Uniform topographic conditions, and

Moderate ground slopes.

In natural subirrigation, water is distributed in a series of ditches about 0.6 to 0.9 metre deep and 0.3 metre wide having vertical sides. These ditches are spaced 45 to 90 metres apart.

Sometimes, when soil conditions are favourable for the production of cash crops (i.e., high-priced crops) on small areas, a pipe distribution system is placed in the soil well below the surface. This method of applying water is known as artificial subirrigation. Soils which permit free lateral movement of water, rapid capillary movement in the root-zone soil, and very slow downward movement of water in the subsoil are very suitable for artificial subirrigation. Thecost of such methods is very high. However, the water consumption is as low as one-third of the surface irrigation methods. The yield also improves. Application efficiency generally varies between 30 and 80 per cent.

Sprinkler Irrigation

Sprinkling is the method of applying water to the soil surface in the form of a spray which is somewhat sililar to rain. In this method, water is sprayed into the air and allowed to fall on the soil surface in a uniform pattern at a rate less than the infiltration rate of the soil. This method started in the beginning of this century and was initially limited to nurseries and orchards. In the beginning, it was used in humid regions as a supplemental method of irrigation. This method is popular in the developed countries and is gaining popularity in the developing countries too.

Rotating sprinkler-head systems are commonly used for sprinkler irrigation. Each rotating sprinkler head applies water to a given area, size of which is governed by the nozzle size and the water pressure. Alternatively, perforated pipe can be used to deliver water through very small holes which are drilled at close intervals along a segment of the circumference of a pipe. The trajectories of these jets provide fairly uniform application of water over a strip of cropland along both sides of the pipe. With the availability of flexible PVC pipes, the sprinkler systems can be made portable too.

Sprinklers have been used on all types of soils on lands of different topography and slopes, and for many crops. The following conditions are favourable for sprinkler irrigation (1):

Very previous soils which do not permit good distribution of water by surface methods,

Lands which have steep slopes and easily erodible soils,

Irrigation channels which are too small to distribute water efficiently by surface irrigation, and

Lands with shallow soils and undulating lands which prevent proper levelling re- quired for surface methods of irrigation.

Besides, the sprinkler system has several features. For example, small amounts of water can be applied easily and frequently by the sprinkler system. Light and frequent irrigations are very useful during the germination of new plants, for shallow-rooted crops and to control soil temperature. Measurement of quantity of water is easier. It causes less interference in cultivation and other farming operations. While sprinkler irrigation reduces percolation losses, it increases evaporation losses. The frequency and intensity of the wind will affect the efficiency of any sprinkler system. Sprinkler application efficiencies should always be more than 75 per cent so that the system is economically viable.

The sprinkler method is replacing the surface/gravity irrigation methods in all developed countries due to its higher water application/use efficiency, less labour requirements, adaptability to hilly terrain, and ability to apply fertilizers in solution..

Trickle Irrigation

Trickle irrigation (also known as drip irrigation) system comprises main line (37.5 mm to 70 mm diameter pipe), submains (25 mm to 37.5 mm diameter pipe), laterals (6 mm to 8 mm diameter pipe), valves (to control the flow), drippers or emitters (to supply water to the plants), pressure gauges, water meters, filters (to remove all debris, sand and clay to reduce clogging of the emitters), pumps, fertiliser tanks, vacuum breakers, and pressure regulators. The drippers are designed to supply water at the desired rate (1 to 10 litres per hour) directly to the soil. Low pressure heads at the emitters are considered adequate as the soil capillary forces cause the emitted water to spread laterally and vertically. Flow is controlled manually or set to automatically either (i) deliver desired amount of water for a predetermined time, or (ii) supply water whenever soil moisture decreases to a predetermined amount. A line sketch of a typical drip irrigation system is shown in Fig. 3.6. Drip irrigation has several advantages. It saves water, enhances plant growth and crop yield, saves labour and energy, controls weed growth, causes no erosion of soil, does not require land preparation, and also improves fertilizer application efficiency. However, this method of irrigation does have some economic and technical limitations as it requires high skill in design, installation, and subsequent operation.

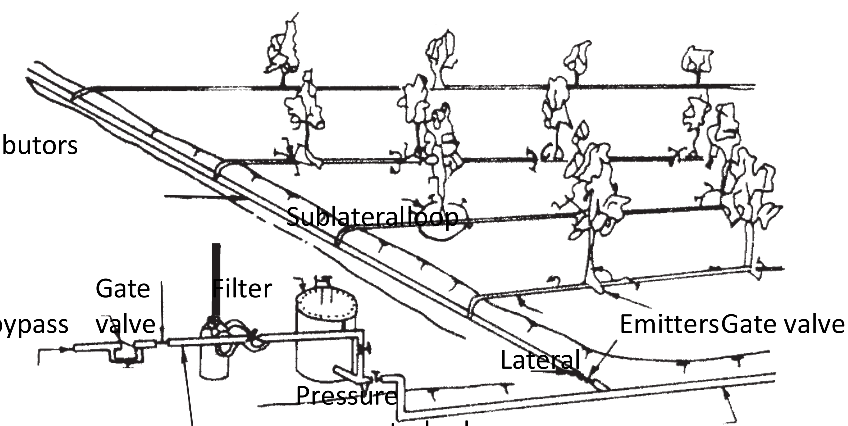
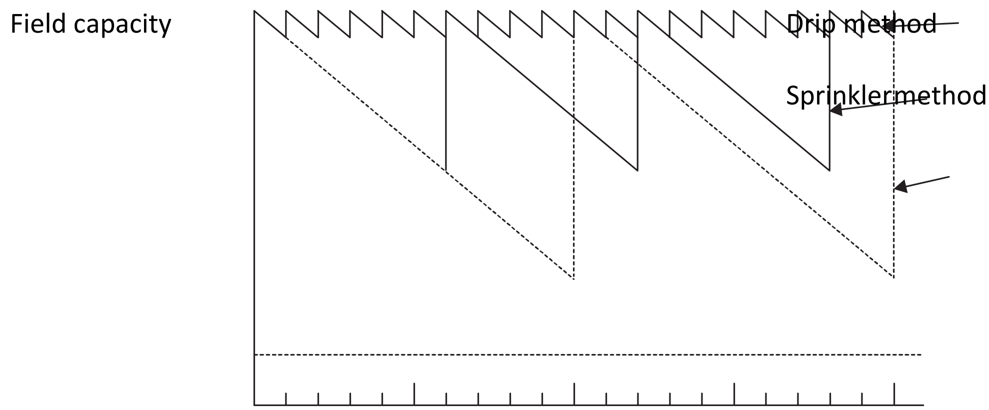


Fig. 3.6 Line sketch of a typical drip irrigation system

Trickle irrigation enables efficient water application in the root zone of small trees and widely spaced plants without wetting the soil where no roots exist. In arid regions, the irrigation efficiency may be as high as 90 per cent and with very good management it may approach the ideal value of 100 per cent. The main reasons for the high efficiency of trickle irrigation are its capability to produce and maintain continuously high soil moisture content in the root zone and the reduction in the growth of weeds (due to limited wet surface area) competing with the crop for water and nutrients. Insect, disease, and fungus problems are also reduced by minimising the wetting of the soil surface.

Due to its ability to maintain a nearly constant soil moisture content in the root zone, Fig. 3.7, trickle irrigation results in better quality and greater crop yields. Fruits which contain considerable moisture at the time of harvesting (such as tomatoes, grapes, berries, etc.) respond very well to trickle irrigation. However, this method is not at all suitable (from practical as well as economic considerations) for closely planted crops such as wheat and other cereal grains.


Field capacity
Drip method
Sprinklermethod

Surfacemethod Moisture content

Wiltingpoint

0 5 10 15 20
Days

Fig. 3.7 Moisture availability for crops in different irrigation methods

One of the major problems of trickle irrigation is the clogging of small conduits and openings in the emitters due to sand and clay particles, debris, chemical precipitates, and organic growth. In trickle irrigation, only a part of the soil is wetted and, hence it must be ensured that the root growth is not restricted. Another problem of trickle irrigation is on accountof the dissolved salt left in the soil as the water is used by the plants. If the rain water flushes the salts near the surface down into the root zone, severe damage to the crop may result. In such situations, application of water by sprinkler or surface irrigation may become necessary.

Because of the obvious advantages of water saving and increased crop yield associated with the drip irrigation, India has embarked on a massive programme for popularising this method. The area under drip irrigation in India is about 71000 hectares against a world total of about 1.8 million hectares (6). The area coverage is the highest in Maharashtra. (about 33000 hectares) followed by Andhra Pradesh and Karnataka. Cost of drip irrigation system in India varies from about Rs. 15000 to 40000 per hectare. The benefit-cost ratio (excluding the benefit of saving in water) for drip irrigation system varies between 1.3 to 2.6. However, for grapes this ratio is much higher and may be as high as 13.

Agriculture
Principles of Agronomy
Soil Management
Women in Agriculture
Value Added in agriculture

Agriculture

Agriculture is the art, science and business of crop production. It encompasses allaspects of crop production, livestock farming, fishery and forestry. Agriculture is the conversion ofsolar energy into the chemical energy. Crop production is the conversion of environmental inputs like solar energy, carbon dioxide, water and nutrients in soil to economic products in the form of human or animal food or industrial raw materials.

Agronomy is derived from the greek words Agros meaning field and nomos meaning manage. It is branch of agricultural science which deals with principles and practices of soil, water,and crop management.

Agronomy deals with methods which provide favorable environment to higher crop productivity.

According to Norman (1980) it is a science of manipulating the crop environment complex with dual aims of improving productivity and gaining a degree of understanding of the process involved.

The recent definition of agronomy is the successful, sustainable, profitable, nutritionally secured, efficient crop production with least or no environmental degradation.

Agriculture is a science of farming. Scientific principles are employed to find ways of making it as efficient possible. Through scientific principles plants and animals are transformed genetically and most favorable environment is provided to harvest higher yields of good quality withleast expense of energy. The scientific principles of various branches viz. soil science, genetics and plant breeding, entomology, plant pathology, microbiology, agricultural engineering etc. were employed in agriculture. Agriculture like any other science is a body of truths synthesized and systematized and arranged in such a way as to show the operation of general laws and principles.

Agriculture is an art which embraces knowledge of the way to perform the operation of a farm in a skillful manner. The physical and mental skills are involved in agriculture. The skills may be acquired through years of experience viz. ploughing, stacking hay bundles, handling animals, sowing, transplanting, driving a tractor etc.

Mental skills are those involved in decision making for example when to plough the land, selectionof appropriate crops, seed selection etc.

Agriculture is a business: agriculture is no longer a way of living or subsistence agriculture where production is intended to meet the home requirements. Agriculture is intended to earn more income. Land, labour, capital are judiciously used. Like in any industry the farming industry should forecast the demand, tailor the production with demand to earn more profits. It involves processing,value addition, transportation, packing, storage in scientific way. Knowledge of employee and employer relationship or human resource management, export and imports, taxation, customs, tariffsand trade are required. All these aspects demand business knowledge in addition to the production and managerial skills. Traditional agriculture is no longer relevant for success in agriculture. Commercial Agriculture or Corporate Farming, Agri-Business Development Corporations demand entrepreneurs in agriculture rather than technologists alone.

Factors affecting crop production

Crop growth is influenced by internal factors and external factors. Internal factors are controlled by the genes and hereditary. External factors are climate, edaphic, biotic, physiographic and anthropic

Climatic factors:

Precipitation occurs in the form of rainfall, snow, hail and fog

Fog consists of water droplets so small that their fall velocity are negligible. Fog particles contact vegetation may adhere, coalesce with other droplets and eventually form a drop which is large enough to fall to ground.

Dew: during night there is loss of heat by radiation. Condensation of water vapour present in the air results in dew. Winter crops are efficient collectors and users of dew

Rainfall is the most important factor affecting the vegetation of place. Most of the crops receive their water supply from rain. The yearly precipitation i.e quantity, intensity and distribution largelyinfluence crop growth.

Low and illdistributed rainfall is most common in dry lands wherein drought tolerant crops are grown.

Heavy and regular rainfall is the common feature in western ghats. High water requiring crops are grown eg coffee, cardmom, pepper, banana etc.

Desert : it is the least rainfall receiving area. Desert grasses and shrubs are common vegetation. Drought: is condition of continuous lack of moisture so serious that crops fail to develop and matureproperly

Adaptation to moisture situations:

Plants assimilate 0.1-0.3 per cent of the water absorbed from the soil. Hydryphytes: aquatic plants which are grown in water. Swamp and bog plants.

They have spongy tissue. Stomata are numerous and located on the upper side of the leaves eg. Waterhyacinth, rice, eelgrass

Mesophytes: most common land plants. Stomata are more confined to or more numerous on the under side of the leaves. Root hairs are abundant. Root length and volume often equal or exceed topgrowth. True mesophytes wilt after loosing 25 per cent of their total water content. Xerophytic- mesophytes wilt after loosing 25 -50 per cent of their total water content.

Xerophytes: these are capable of enduring prolonged drought without injury. They will grow in a substrate which is depleted of water for growthto a depth of 20 to 25cm. They have modifications viz. reduced stomata, respiration, transpiration. In cati the carbon dioxide released during respiration stays in chlorenchyma tissue. It is reassimilated. Hydrophyllic cholloids in certain wheat varieties induce drought resistance. Due to bound water. During drought the expend not more than 2-10 per cent of the water absorbed.

Effects of excessive moisture:

Limit oxygen supply

Formation of toxic substances
Leaching of nutrients eg nitrates
Detrimental to germination, flowering, pollination and fruiting
Continued turgidity, low transpiration,
Disease incidence viz. rust, mildews,
Curing and storing of produce hindered

Temperature
Temperature influences the following plant processesBiochemical reactions
Uptake of carbon dioxide Production of chloroplasts Production of growth substancesPhotosynthesis
Dry matter productionGermination
Leaf initiation Leaf emergenceLeaf expansion

Flowering
Spikelet development

Grain developmentYield

Various biochemical process are associated with photosynthesis are controlled by temperature. Under high temperature photosynthesis become heat inactivated. Retardation of growth, adverse effects on fertilization at temperature below leathel limit.

Temperature is the measure of heat energy. The range of maximum growth for most agricultural plants I between 15-40^{o} C. Temperature of a place is largely determined by the distance from the equator (latitude) and altitude. Vegetation is classified into four classes based on the temperature requirement.

Megatherms: Equatorial and tropical, high temperature requiring eg rice, rubber, banana Mesotherms: Tropical and subtropical, high temperature alternated with low temperature, eg maize,sorghum

Microtherms: Temperate and high altitude plants, low temperature requiring, wheat, oats, potatoHekistotherms: Very low temperature requiring eg. Pines, spruce

Every plant community has its own minimum, optimum and maximum temperature known ascardinal points

Cordinal temperature of certain plants

Crops	Minimum	Optimum	Maximum
Wheat	3.8-4.4	24.9	29.9-32.2
Barley	3.8-4.4	19.9	27.7-29.9
Oats	3.8-5.0	24.9	29.9
Maize	7.7-9.9	31.6	39.9 -43.8
Sorghum	7.7-9.9	31.6	39.9
Rice	9.9-11.6	32.2	36.1-38.3
Tobacco	12.7-13.8	27.7	34.9

The minimum daily mean temperature at planting time for potato 7^{o}C , corn 14 ^{o}C, Cotton 17 ^{o}C. minimum temperature for germination of maize 4 ^{o}C, sorghum 9 ^{o}C, rice 9 ^{o}C,

Minimum temperature for growth initiation for sorghum 15-18 ^{o}C, optimum temperature for mosttemperate crops is 24-29 ^{o}C, maximum temperature 35-41 ^{o}C

Cool season crops fail to grow at an average temperature of 30-38 ^{o}C,

Wheat, potato, barley, oats require max temp of 30-38 ^{o}C, min 0-5 ^{o}C, optimum temp 20-30 ^{o}C

Warm season crops viz sorghum, maize, sugarcane, groundnut, redgram, cowpea, pearlmillet require maximum temp of 40-50 ^{o}C, minmum 15-20 ^{o}C, optimum temp. 30-38 ^{o}C.

Temperature effects on plants

Chilling injury: some plants growing in hot climate if exposed to low temperature (above freezing point) express chlorotic condition or bands on the leaves. eg sorghum, sugarcane, maize when exposed for 60hrs at 2-4 ^{o}C on the other hand cold loving plants viz. potato suflower tomato are unaffected.

Freezing injury: this is generally caused in plants growing in temperate regions. Water is frozen into ice crystals in the intercellular spaces. Frost damage in potato, tea are common

Suffocation: during winter ice or snow forma thick cover over the ground and the crop suffers for want of oxygen.

Heaving: injury to plants caused by a lifting upward of the plant along with the soil from its normalposition in temperate regions where snow fall is common

Heat injury: very high temperature often stops growth. The plant faces incipient starvation due to high respiration rates. The plant is stunted. If such condition persists for a long period the plant is killed. Sterility of plants, young seedlings are killed, defoliation premature dropping of fruits are theadverse effects of high temperature

Vernalization: some plants require cold stimulus before they come for flowering. The cold treatment given to the sprouting seeds to effect the flowering is known as vernalization. Lysenko a Russian scientist proposed the incubation of sprouted seeds at temperature just above 0 ^{o}C for 2-3 weeks before sowing.

Thermoperiodism

The response of plants to rhythmic fluctuations in temperature is known as thermoperiodism. A number of physiological process viz germination, stem elongation, fruiting, floral development andincrease in frost hardiness may proceed at most satisfactory rate under rhythem of alternating temperature.

Humidity: refers the the invisible water vapour present in the atmosphere. The air is said to be saturated when it holds maximum amount of water vapour at a particular temperature. The humidity in atmosphere is termed as relative humidity (RH)

Importance of RH

It is related to water relations in plant. Directly related to evapotranspiration Indirectly related to leaf growth

Photosynthesis: when RH is low transpiration increases causing water deficits in the plant. Water deficit causes partial closer of the stomata and increases the mesophyll resistance thereby blocking the entry of carbon dioxide.

Pollination: when RH is high pollen dispersal from the anthers. Seed set is more at moderate RH thanat high RH.

Pests: incidence of pests and diseases is high under high humidity. Under high RH fungal spores will germinate easily. Spread of blight disease of potato and tea is more rapid uner high RH. Aphid andjassid incidence is more under high RH.

Wind: the turbulence in the atmosphere is termed as wind. Moderate winds are essential for pollination, exchange of carbon dioxide in the canopy, winds also cause rains.

Adverse effects of wind:

High wind velocity increases transpiration, accelerates the desiccation of crop, reduce plant height, normal form and position of the shoot is permanently deformed when developing shoot is continuously exposed to wind from particular direction. Lodging of field crop, tearing of leaves, dropping of fruits, grain shedding, soil erosion, root exposure in deserts, spread of insect pests, spores of fungi. Wind also alters the balance fo hormones in plants wind increases ethylene production in barley and rice. Wind increases gibberilic acid content of roots and shoots in rice. Nitrogen concentration in both barley and rice increases with increase in wind speed.

Atmospheric gases:

Nitrogen of the atmosphere is directly used by symbiotic and asymbiotic nitrogen fixers. Carbon dioxide used by the plants during photosynthesis. Oxygen for respiration. The concentration of nitrogen, carbon dioxide and oxygen in air is 78.19, 0.03 and 20.95 per cent respectively. Sulphur dioxide and nitric oxide in atmosphere reach soil during rains improve soil fertility. Carbon dioxide enrichment of canopy improves crop productivity. On the other hand increase in concentratin of carbon dioxide, methane, nitrous oxide and fluorochlorocarbons in the atmosphere deplete ozone layer causing ultraviolet rays to reach earth and causes global warming.

Light:

Light is one of the most important factors influencing many vital plant processes.

Light is required for synthesis of chlorophyll pigment. Chlorophyll pigment is capable of absorbing radiant energy and converting it into potential chemical energy viz. carbohydrates through the process called photosynthesis. The photosynthesis is directly proportional to the amount of light. Other processes like seed germination, leaf expansion, growth of stem and shoot, production of tillers, branches, flowering, fruiting, root development, growth movements in plants.

Under low light intensity plants grow tall with weak stem which may cause lodging.

Plants obtain light from solar radiation which is the source for light and heat. Light efficiency in plants is less than 2 per cent.

The quality (wavelength and colour) the quantity (the intensity and duration of exposure to light) greatly influence the plant growth.

Quality refers to the wave length and colour. Of the total range of electromagnetic wavelengths in the solar spectrum, light or the luminous energy includes wavelengths between 400-750mμ (milli microns) or nm (nanometers). Quantity of light measured in g cal/m^2 /year. Lux: the light intensityfrom a standard candle at one m distance I a metre candle or lux

Colour	Wave Length	Effect on plants
Ultraviolet	<390 mμ	X rays and gamma rays. Very detrimental for growth accounts 0-4 %

Violet	400-435 mμ	Favorable for plant growth, Phototropism
Blue	435-490 mμ	Favorable for plant growth. Photosynthesis, blue is more efficient than red.
Green	490-574 mμ	Carbon dioxide assimilation
Yellow	574-595 mμ	Carbon dioxide assimilation
Orange	595-626 mμ	Carbon dioxide assimilation
Red	626-750 mμ	Carbon dioxide assimilation More favorable for plant growth
Infrared	>750mμ	Temperature will increase

Ecology, crop distribution and factors affecting crop distribution, adaptation of cultivatedplants

Numerous investigations on soil, plant breeding, choice of species, introduction of new crops, tailoring agronomic requirements for crop production etc. are related to ecology.

Ecology: the term ecology is derived form the Greek word Oikos meaning house abode or dwelling. The term was first introduced by E. Haeckel. It is also called environmental biology. It deals with the study of plants/animals in relation to their environments. According to Odum (1969) it is the study ofinterrelationships between organisms and environment. Recently functional interrelationships or physiological relationships are considered.
Environment implies
Climatic factors- temperature, precipitation, atmospheric gases Edaphic factors- parent material, soil
Biotic factors- other living organisms
Social factors- policies, restrictions, food habit of population, profitability of particular crop etc.

Billings (1952) introduced two more environmental factors
Geographic- soil erosion and deposition, topography, gravity, volcanismsPyric- fire
Plant ecology deals with plants in relation to their environment. There are two divisions under this **Autoecology** deals with the ecology of individual species and its population including the effect of other organisms and environmental conditions on every stage of its life cycle.
Synecology deals with the ecology of plant communities. It involves the study of structure, nature, organization and development of plant communities.
Ecological crop geography is branch of crop ecology which deals with the broad spacial
distribution of crop plants and the rationale of such distribution in terms of physical and socioeconomic environment influencing the production of crops.
Agroecology was proposed by Bensin (1930) which deals with the detailed study of commerciallyimportant crop plants by the use of ecological methods.
Environment is the sum total of effective conditions under which a plant community lives (Tasley,1926).
Ecosystem
System means a unified whole made of regularly interacting or independent components.
Plants like animals do not live independently in nature. They are associated in biotic communities. The biotic community, the functional unit of a habitat which is held together and is inseparable by its members, its dynamics is known as ecosystem. In a ecosystem the living organisms and nonliving entities are inseparably related and continuously interacting. In an ecosystem there is exchange of materials between living and nonliving parts and one living community depends on the other for itssurvival. The components of the ecosystem according to Odum (1959) are abiotic substances, producers, consumers (heterotrophic animals) decomposers (heterotrophic bacteria, fungi).

Adaptation is any feature of an organism which has survival value under the existing condition of its habitat.
The basic of successful crop production is the selection of a adapted plants species and varieties. Plants may have morphological adaptation or physiological adaptation or both for the existing environment.

Morphological adaptations eg.Reduced leaf size, leaf number, number of stomata, depth ofthe root system, waxy coatings, spines *etc.* **Physiological adaptations** eg. presence of arenchyma tissue in aquatic plants, resistance topest and diseases *etc.*

The adaptation of crop plants to climate, soil, and economic situation determine the desirability ofgrowing any crop in a particular region

Crop Distribution

Principles of plant distribution:

Environmental factors greatly influence the natural distribution of plants. Plant geographer Good (1931) formulated principles of plant distribution.

The plant distribution is primarily controlled by the distribution of climatic conditions

The plant distribution is secondarily controlled by the distribution of edaphic factors

The great movements of floras have takenplace in the past and are still continuing (succession offloras as evidenced by the fossil records).

That the species movement (plant migration) is brought about by the transport of individual plantsduring their motile dispersal phases.

That there has been great variation and oscillation in climate, especially at higher latitudes, during the geological history of angiosperms.

That at least some, and probably considerable, variation has occurred in the relative distribution andoutline of land and sea during the history of angiosperms.

The changes in the environment particularly climate resulted in plant movement and migration. Thiswas explained by the Good's concept - specific tolerance according to which

Each and every plant species is able to exist and reproduce successfully only within a definite range of climatic and edaphic conditions. This range represents the tolerance of the species to external conditions

The tolerance of a species is a specific character subject to the laws and process of organic evolutionin the same ways as is its morphological characters, but the two are not necessarily linked.

Change in tolerance may or may not be accompanied by morphological change, and vice versa.

Morphologically similar species may show wide differences in tolerance and species with similar tolerance may show little morphological similarity. The relative distribution of species with similar tolerance is finally determined by the result of competition between them.

The range of tolerance of any larger taxonomic unit is the sum of (or total range or extent of) the ranges of tolerance of its constituent species.

Shelford (1913) proposed a general law of tolerance

Organisms with wide ranges of tolerance for all factors of the environment are likely to be widely distributed

Organisms may have a wide range of tolerance for one factor and narrow range for another. Whenthe conditions are not optimum for one factor the limit of tolerance may be reduced with respect to another factor.

The period of reproduction usually is critical when environmental factors are likely to be limiting.Mason (1936) added that

The extremes of climatic conditions are more significant than the means, which emphasize theperiphery, are limits of range where extremes of climatic factors are most likely to be limiting

Both dispersal and establishment are essential for plant migration

The tolerance theory should emphasize the factor function relationship.

During the life cycle of the plants certain critical phases have narrow tolerance rangeCain (1944) further strengthened the tolerance theory by adding

That the biotic factors may be important,

That the environment is holocoenotic and

That the tolerance has a genetic basis

Biotic factors exert direct influences on plant distribution eg. obligate insect pollination, seed dissemination, grazing by livestock

Billings (1952) stated holocoenotic concept for plant distribution. He emphasized the importance oftolerance of plants to components of climatic, edaphic, and biotic factors for plant distribution.

Climatic- fire, water, wind pressure, atmospheric composition, cosmic radiation, solar radiation, terrestrial radiation, temperature

Edaphic- soil, parent material, gravity, rotational forces, topography andgeographic position etc.

Biotic – man, animals, other plants,

Limiting factors play a significant role in the distribution of plants

Law of Minimum (Liebig, 1840) – the growth of the plants is dependent on the amount of food stuff (or element) presented to it in minimal quantity

Taylor (1934) included environmental factors in addition to the nutrients.The most critical season of the year, the most critical year of the climatic cycle and the critical stagesof development (germination, anthesis) .

Blackman (1905) developed the "Theory of optima and limiting factors."

– according to which "when a process was conditioned to its rapidity by a number of factors, the rateof the process was limited by the pace of the slowest factors".

Lundegardh (1931) emphasized as a factor increases in intensity itsrelative effect on the plant growth decreases. This principle is sometimes called "Law of relativity"

Livingston and shreve (1921) proposed the concept of "Physiological limits" according to which for every vital function there is maximum and aminimum zero point with respect to any conditioning factor, beyond which the function ceases. Further for every distinct climatic area there appears to be a corresponding type of vegetation and this principle is probably ofprimary importance in the study of plant distribution.

Odum (1969) attempted to combine these three theories. Plants appear tobe controlled by three forces

Quantity and variability of materials for which there are minimal requirements

Physical factors which are critical

Limits of tolerance of the plants themselves to these and other factors of the environment.

Cain (1944) who noted that "The capacity of the species to tolerate or respond to its environment is governed by the laws of evolution and genetics, and the range of tolerance is the direct result of the diversity of the species".

Centers of origin of cultivated plants

The process of cultivation itself improved the plants taken from wild. de Candole (1882) tried totrace the ancestors of the cultivated plants by using two criteria

Occurrence of a given cultivated plant in a locality where it also grows wild or where wild relativeswere found

Using information from archaeology, history and linguistic evidences

According to Mendel (1965) the present day cultivated plants originated by hybridization andselection.

Vavilov (1951) established the principle that "the distribution of plant species on the earth is not uniform. The success attained by Vavilov in locating the principal geographic centres of origin ofcultivated species may be attributed to

extensive cultivation of cultivated plants

Thoroughly studied the collected plants

Vavilov recognized **eight primary centres** of origin based on the

Diversity of heritable forms

Based on certain endemic varietal characters

Presence of closely related wild or cultivated forms

Presence of genetically dominant characters generally in the core of the centres of origin

Archaeological, historical and linguistic evidences

A great diversity of species was found which is of later development in the region other than primarycentre known as the **secondary centre** (Harlon 1951).

Agro – ecological groups or gene microcenters are the smaller centres within a main centre(Vavilov).

Soil Management

Importance:

Successful farming concerns the appropriate management of soil, plants and environment in such a way that a maximum return can be obtained not only in a season or year but also over centuries. The physical, chemical and biological properties of soil and their modifying factors regulate the present and future state of soils, the source of infinite varieties of life. The most important consideration in soil management is the correct application of the relationships among the soil, the environment andthe crops to be grown.

There is no substitute for soil for crop production. The soil is the precious natural gift. The land is not unlimited. The per capita availability of is 0.13ha. There is no scope for horizontal expansion butonly way is to increase productivity through judicious soil management. Soil is not inherited from out ancestors but borrowed from our future generations. It should be returned without impairing thequality.

The problem of soil management vary according to soils and their situation in the land, the climatic conditions, biotic influences and crops to be grown, yet there are fundamental factors which govern the choice of a suitable soil management practice.

What are good soil management practices?

Good tilth is the first feature of good soil management. It means a suitable physical condition of the soil and implies in addition a satisfactory regulation of soil moisture and air.

The maintenance of soil organic matter which encourages granulation is an important consideration of good tilth.

Tillage operations and timings should be adjusted as to cause the minimum destruction of soil aggregates. Good tilth minimizes erosion hazards.

The choice and sequence of adaptable crops or crop rotation are other very important considerations. These are related to climate, particularly rainfall and its pattern of distribution and the characteristics of the soil profile, including drainage and extent and duration of available soil moisture. A proper sequence of crop varieties greatly influences soil conditions. It is more realistic to evolve cropping patterns and land management practices according to land capability. Cropping patterns chosen and management practices adopted should aim at soil and moisture conservation for efficient nutrient and moisture utilization.

In irrigated areas, special management practices become necessary to avoid salinity, alkalinity, waterlogging, leaching and the loss of plant nutrients. In rainfed areas special management practices include improving soil conditions to receive, retain and release more soil moisture., harvesting water to use as life saving irrigation or extending the cropping season when there is insufficient rainfall for raising crops, protecting the soil from degradation both in cropped and bare fields. Land shaping and leveling mulching and the use of wind brakes and vegetative cover are the other major aspects.

The productive capacity of the soil should never be allowed to diminish, but rather should be improved and maintained by providing adequate organic manures and plant nutrients through fertilizers and by including legumes in the rotation and the use of biofertilizers. Similarly the provision of irrigation facilities in semi arid and arid areas, the adoption of different remedial measures against excessive salinity and alkalinity or acidity in humid areas, the use of specific soil amendments to correct imbalances of plant nutrients and the application of micronutrients where they are deficient.

Economic plant protection measures against pests, pathogens and parasites including weeds should form part of the management practices in the cropping system. This can be achieved by following recommended cultural practices or by application of pesticides/fungicide/herbicides.

The management practices adopted should be economically profitable and emphasis should be laid on maximizing sustained income rather than yields for the time being. An integrated land plan including all the above points and economically profitable should be developed for individual situations.

Results of bad soil management

- Soil erosion : tillage practices should be oriented to conserve soil and water
- Soil exhaustion: avoid continuous cultivation of exhaustive crops, regular use of organic manures and fertilizers
- Salt accumulation: poor drainage, use of poor quality water, toxic substances from synthetic fertilizers and agro industrial wastes
- Infestation with perennial weeds:
- Structure of the soil is spoiled
- Lower productivity and profits

Requirements of ideal seed bed

An ideal seed bed is one which sustains all stages of crop growth and development starting from germination and emergence to maturity without great deterioration and depletion.

Seed bed should be free from large clods, crop residues and established weeds

Should be properly leveled and of a desirable physical state with the correct moisture content for good growth and yield of crop

There must be provision for adequate irrigation channels and drainage
The soil should be supplied with basal dose of manures and fertilizers
Stiff stubbles, stalks and other readily decomposable organic matter should be removed otherwise they may invite infestation of pests and pathogens.
Granulation should be retained. Avoid excessive tillage than the required. Poor granulationencourages soil erosion.
Depth of tillage should be need based
There must be adequate moisture and air supply in the seed bed.
Rough seed beds are favored for winter and large seeded crop Firmer and shallower seedbed for fineseeded crop. For soil moisture conservation fine seed beds during summer in light soils is essential.
Unweathered cloddy soil should not be brought to the surface by deep tillage.
Loose granular seed bed for drylands while puddle seed bed for wet lands
Seed beds should be prepared quickly to avoid loss of moisture. Soil moisture loss can be prevented by breaking and sealing of pores and also by reducing exposed area by leveling by planking or harrowing
Manures and fertilizers applied should be thoroughly mixed into the soil.
Proper land shaping and configuration is essential for soil moisture conservation for example ridge and furrow, broad bed and furrow or flat bed depending upon the situation and crop requirement.

Good soil management practices

- Good tilth
- Control of weeds
- Maintenance of adequate levels of organic matter
- Adequate supply of plant nutrients
- Control of pests and pathogens
- Adoption of soil and water conservation practices
- Adoption of suitable crop rotation
- Providing drainage
- Alleviating the soil from excess salts, acidity, toxic substances.

Tillage in the physical manipulation of the soil with tools and implements to result in goodtilth, for better germination and subsequent growth of crops.
Tilth is the physical condition of the soil resulting from tillage. Soil is said to be in good tilth when itis mellow friable and adequately aerated.
Tilth is dynamic. Mechanical forces may change the roughness of the soil surface, the total porosityand bulk density of the tilled layer and the aggregate or clod size. Heavy rainfall and high velocity wind or water destroys the tilt for soil moisture conservation h.
The action of wetting and drying, freezing and thawing regenerates desirable tilth.
The roughness of the soil surface is an index of the amount of water that can be stored in soil depressions and may also be related to the resistance of the soil surface to sealing. Roughness may berelated to rates of evaporation and transfer of heat and air between the soil and atmosphere.

Objectives of tillage

The primary purpose of tillage is often to reduce the aggregate or clod size. Aggregates must be small enough to achieve good contact between the roots of the seedlings and soil in order to prevent drying of the soil, to provide sufficient soil solution, aeration. Yet the aggregates should not be so finely divided as to encourage sever crusting when dry.
To provide suitable seed bed for sowing the seeds necessary for germination and emergence of seedling or for transplanting seedlings or planting materials. Fine seed bed is desirable for smaller sized seeds while rough seed bed is enough for larger seeds.
To control of weeds, pests and diseases
The residues of the previous crop are incorporated into the soil.

To improve the physical condition of the soil viz. structure (development of granular structure), porosity and bulk density, aeration, water holding capacity, infiltration and reduction in run off. Thehard pans are broken during ploughing which facilitate greater infiltration and root penetration into deeper layers. The positive influence of tillage on these properties will ultimately reduce soil erosion.

To improve the nutrient availability by physical process

To hasten the chemical and biological processes in the soil. This in turn has impact on the activity ofmicroorganisms and organic matter decomposition.

Effect of tillage on soil temperature, soil moisture and root penetration

Soil temperature: tillage influences soil thermal properties. Loosening the soil decreases the heat capacity because of larger volume fraction of soil air. Thermal conductivity is decreased with decreasing fraction of solids and increased with increasing water content. Loosening the soil often results in a larger difference between maximum and minimum temperatures but this may be comparatively less than air temperature. Soil temperature affects germination, nutrient availability, plant development, grain yield and nutrient content of grains.

Soil moisture: tillage may have marked influence on soil moisture through its effect on infiltration, run off, temporary surface storage internal storage and availability to plants. Roughness of soil surface is an index of the amount of water that can be stored in depressions and may also related toresistance to sealing. Tillage creates uneven micro-relief which can store considerable water in the small depressions for later infiltration. Once water enters the soil its rate of movement will depend on the internal transmission characteristics of the profile. Increased porosity from tillage may act asan important reservoir for temporary storage of water during rains. The amount water retained by asoil may be influenced by the soil density and aggregate size. Ploughing or loosening a dense soil provides pores which can store water and may increase the storage capacity. In loose soils when surface soil is dry water vapor moves to surface. If there is rainfall infiltration is higher. In compact soil at relatively higher moisture content there will be greater conductance of water to theevaporating surface.

Soil aeration: Soil aeration is the mechanism of exchange of oxygen and carbon dioxide betweensoil pore space and the atmosphere in order to prevent the deficiency of oxygen and toxicity ofcarbon dioxide. Respiration by plant roots and microorganisms depletes oxygen and releases carbon dioxide and minute quantities of other gases into the soil atmosphere. The amount of carbon dioxide varies widely depending on temperature, organic matter and microbial respiration. A constant inflow of oxygen and out flow of carbon dioxide is essential for plant growth. Loosening a dense soilfacilitates exchange of gases between soil air and atmosphere. Air filled pores commonly fluctuatefrom 15 to 30 per cent of the total volume depending upon water content, soil density and structure. <u>Causes of poor soil aeration</u>

Compact soil

Water logging

Soils having excessive amount of readily decomposable organic matter

Harmful effects of poor aeration

Microbial activity is reduced, slow decomposition of organic matter

Rhizobium cannot symbiotically fix atmospheric nitrogen

Abnormal development of roots ex: sugarbeet and carrot

Anaerobic bacteria decompose soil organic matter in the complete absence of oxygen releasetoxic substances like sulphides which are harmful to crop plants

Root penetration: plant roots can exert tremendous forces in penetrating the soil. In packed soils pore size may be too small for root tips to enter. Critical pore size varies with kind of plant, but thereare pore diameters below which roots may not penetrate. Pore rigidity as well as size will affect theroot penetration.

Types of tillage

Clean tillage / conventional tillage/ traditional tillage: is one wherein 100 per cent of the top soilis mixed or inverted.

Conventional tillage has been defined as combined primary and secondary tillage operationsperformed in preparing the seed bed.

With clean tillage all the plant residues are removed and buried. The growth of weeds is prevented.

Suitability

Adopted in Class I lands

Advanatages

Weeds are efficiently controlled

Crop residues are thoroughly incorporated

Better microbial activity

Disadvantages:
More energy requirement
Greater loss of soil moisture
Formation of hard pan
Surface soil is more prone to erosion
Modern concepts of tillage
Minimum tillage:
Zero tillage
Conservation tillage
Stubble mulch tillage
Blind tillage
Minimum tillage may be defined as a group of soil preparation methods for planting in which the number of tillage operations over the field is less than conventional tillage. This can be achieved byomitting the tillage operations which do not give much benefit when the compared to the cost. For example combining seeding and fertilizer application, row zone tillage, plough plant tillage, wheel track planting
Suitability
Medium textured soils
Advantages
Reduced soil compaction
Better soil conservation
Energy requirement is less
Reduced labour and machinery
Time saving
Disadvantages
Low seed germination and establishment. Difficulty in sowing
Nodulation is adversely affected
Use of herbicides is indispensable
Decomposition of organic matter is slow
Perennial weeds may become dominant
Zero tillage is an extreme form of minimum tillage. Primary tillage is completely avoided and the secondary tillage is restricted to seedbed preparation in the row zone only. Planting is done in previously unprepared soil by opening a narrow slot or trench or band only of required width and depth for sowing and covering the seed or seedling.
Weeds are taken care by using broad spectrum nonselective and non-persistent herbicide before sowing subsequent to sowing by using selective and persistent herbicides.

Till planting is one of the zero tillage method in which heavy machineries are used to clean a narrowstrip over the crop row. Then a narrow band of soil is opened. Seeds are placed and covered.
Advantages
Saving in energy, labour and time
Reduced compaction
Increased earthworm activity and soil organic matter
Disadvantages
Difficult establish optimum crop stand
Mineralization of soil organic matter is slow
Nitrogen requirement of crops is high
Build up of perennial weeds and pests
Conservation tillage is the tillage operation performed to reduce soil erosion and to conserve soil moisture is referred as conservation tillage. This is achieved by covering the soil at least by 30 per cent of the surface by the crop residue. Conservation tillage operations include reduced tillage operations like minimum tillage, no tillage, mulch tillage.
Stubble mulch tillage is method of tillage in which a mulch crop grown during the fallow period orthe stubbles of the previous crop are uprooted and brought to the surface and spread during tillage operation. The main objective is to protect the soil from erosion.
Suitable in sloppy areas and dry lands

Advantages
Reduced loss of soil and water
Improved organic matter content
Disadvantages
Less effective weed control
Stubbles interfere in sowing operation
Blind tillage is the tillage of the soil after sowing a crop either before the crop plants emerges or while they are in early stages of growth. It is extensively employed in sorghum and drilled paddy where emergence of crop seedlings is hindered by soil crust formation on receipt of rain or by irrigation immediately after sowing. Shallow harrowing with entire blade harrow without disturbingthe emerging crop seedlings will loosen the soil crust and help in emergence of seedlings. Generallyweed seedlings emerge within two to three days after sowing while many cereals take 7 to 8 days for seedling emergence. By blind tillage weeds are killed at their early stages.
Planting materials
Plants are propagated by asexual/ vegetative method and sexual methodVegetative method: parts of the plant other than the seed is used
Apomictic embryos: development of an embryo from cell other than a fertilized egg. eg citrus
By runners or stolons: long slender side branches grow above ground. At each node shoots and rootsdevelop ex: straw berry, white clover
Layerings: apple, pomegranate
Suckers: is a laterally growing subterranean off shoot from the base of the main stem of a plant ex:cardamom, banana
By separation
Bulbs : onion, garlic, cloves
Corms: swollen base of a stem axis with distinct nodes and internodes ex: ginger, turmeric, gladiolus
By division: specialized stem structures ex: pineapple, canna
Offsets: pseudo bulb is long and jointed with many nodes. Offsets are developed at these nodes.Roots develop from the base.
Tubers: potato
Tuberous roots: sweet potato
Crowns: straw berry
Cuttings: red raspberry, lemon,
Stem cutting: rose, sugarcane, drum stick
Rooted slips: basal two are three internodes of the stem with a few roots ex: paragrass, guinea grass,hybrid napier
Leaf cutting: bignonia
Grafting : mango
Budding: rose, mango, grape
Micro propagation:
Meristem cultures: orchids
Tissue culture: tobacco, potato, banana

Seed is a plant embryo in a dormant state, surrounded by a food supply and protective outer skin orseed coat. Seed is produced after flower has been fertilized. Seed is a fertilized ovule.
Seed has three parts
Cotyledons/ embryonic leaves
Embryo: from which growth commences
Seed coat: protective coveringEmbryo has two parts:
Radicle: This grows downwards and gives rise to root of new seedling.

Plumule: grows upwards gives rise to shoot or stem. Always the radicle starts growing first and thenthe plumule.
Hilum: the scar on the groove of the seed by which the seed is attached to the pod
Micropile: minute opening through which the seed absorbs water.

Characters of good seed

Seeds must be true to type they must belong to the proper variety or strain of the crop which isproposed to grow
Seeds must be healthy , free from inhert materials, weed seeds or other crop seeds
Seeds must be uniform in size, shape and colour
They must be viable, high germination percentage
Free from pests and disease causing organisms
The seed packet must have label

Four generation scheme is in effect in producing certified seeds
Breeder seed or nucleus seed : Produced by the originating or sponsoring breeder or institution so grown and managed as to maintain the cultivar characteristics
Foundation seed: produced fro fields planted with breeder seed and so handled as to maintain the genetic identity and purity of the cultivar. It is the source of all certified seed either directly or through registered class
Registered seed: the progeny of foundation seed so handled as to maintain genetic identity. Registered seed is of a quality suitable for the production of certified seed
Certified seed: the progeny of foundation or registered seed that has been handled so as to maintain satisfactory genetic purity and that has been approved and certified by the certifying agency.
Depth of sowing:
If sown shallow the top layer of the soil desiccates very quickly in dry weather deep sown seeds take more time for emergence of seedlings. The food stored in the seed may be exhausted for hypocotyls growth before the plumule reaches land surface. There may be failure of emergence.
Big seeds : French bean, maize, Bengal gram sown to a depth of 7.5 cmMedium seeds : upto 5 cm deep
If shallow sown seeds are picked by the birds,

Women in Agriculture

Some historians believe that it was Women who first domesticated crop plants and thereby initiated the art and science of farming. While men went out hunting in search of food, women started gathering seeds from the native flora and began cultivatingthose of interest.

Women play a significant and crucial role in agril. development and allied fields (crop production, livestock production, horticulture, post-harvest operations). The extent of women's involvement in agriculture varies greatly from region to region, among castes, classes. But there is hardly any activity in agril. production, except ploughing in which women are not actively involved.

Women constitute about one half of the nation's population. Seventyfour per cent of the entire female working force is engaged in agril. operations. About 60-70% of agril. operations are handled exclusively by farm women.

Farm women – an adult female actively involved in farming operation. Women participate in several activities such as seeding, transplanting, weeding, fertilizer application, plant protection, thinning, harvesting, processing, winnowing, cleaning, storing, looking after the animals, kitchen gardening etc. Several of these operation are exclusively carried out by women only. Thus, by participating in these agril. activities they directly or indirectly influence agriculture and animal husbandry development. About two-third of human labour work hours in agriculture is done by the female labour. Accordingly to FAO, "Women produce between 60 & 80% of the food in most developing countries and are responsible for half of the world's food production.

Multi-dimensional role of women

Woman performs 4 district function – mother, wife, home maker and worker

Agriculture : sowing, transplanting, weeding, irrigation, fertilizer application, plant protection, harvesting, winnowing, storing etc.

Domestic: Cooking, child rearing, water collection, fuel wood gathering,household maintenance.

Allied activities: Cattle management, fodder collection, milking etc.

Mainly rural women are engaged in agril. activities in three different ways dependingon socio-economic status of their family and regional factors. They work as:

Paid labourers (49%)

Cultivator doing labour on their own land (17%)

Managers of certain aspects of argil production by way of labour supervision andthe participation in post harvest operations (10%)

Works in her own farms and also as a wage laborer (24%).

Share of farm women in agriculture operations

Activity	Involvement(%)
1. Land preparation	32
2. Seed cleaning and sowing	80
3. Inter-cultivation activities	86

Harvesting – reaping, winnowing, drying, cleaning, storage 84

In the peak season, an active farmwoman spends 5 to 9 hours/day on the farm.

Agriculture and allied activities almost take the equal time and energy at par with household activities (7 hr. 55 min.).

Size of farm, farm commodity, marital status, control of land, children on the farm, husband 's off farm work, education and experience in farming affect the extent to which women are involved in tasks. In diversified farm, women have the highest level ofinvolvement.

Conditions and problems of women agricultural labourer

Agril. wages and family income of agril. workers are very low in India.

The women labourers are discriminated in wage payment, though the constitution of India provided equal rights and privileges for men & women. Female labourers are paid less when compared to men. Gendered division of labour on farms influences the types of farm tasks performed and extent of women's involvement in farming.
Invisibility of farm women's work.
Women agril. labourers face the problems of unemployment and under- employment for a substantial part of the year, because there is no work on the farm, employed only for a part of the year.
No provision for fixation of hours of work. At the time of sowing & harvesting, they have to work on the farm from dawn to dask, since they are employed on a daily basis. Long working hours under adverse climatic conditions.
No leave/other benefits, no sick leave.
No social security.
No clear cut distinction among operations.
Lack of appreciation for good work
Not having the liberty to take individual decisions at the works – spot.
The wager not being sufficient to provide them with enough food leading to poor nutrition and health condition

<u>Stress factors</u>
Women are often sandwitched between caring for elderly and children. Thiscan lead to role overload and increased stress.
Farm wives keep the farm business and family life running smoothly
Added economic stress along with role overload is consistent with the farmfamily stress. These multiple roles may add to the risk.
Women have the added burden of house hold responsibilities and may feel stress because they are not able to maintain the household in the way that they would like.
Physical factors such as age, physical stature, and physical health status (osteoporosis , vision problems), fatigue and stress.
Female agril. labourers do not enjoy any maternity leave and do not get properrest after child birth.
Meager wages, long hours of work, hazardous work. Lower wages than men, over time work, walking long distance to the work spot.

<u>Measures adopted by the Govt.</u>
Minimum wages Act.
Abolition of bonded labourers
Providing land to landless labourers
Provision of housing sites
Special schemes for providing employment
Rural workers programme (RWP),
Employment guarantee scheme (EGS)
Food for work programme (FWP)
Notional Rural Employment Programme (NREP)
Rural Landless Employment Guarantee Programme (RLEGP)
Providing employment during off-season.
Development of women and children in Rural areas (DWCRA) .

Value Addition in Agriculture
The profits on agricultural commodities have greatly diminished. The cost of production has increased faster than the market price of the outputs. There is a need toincrease the farmers' earning through value addition.
Value addition to raw food material in Nepal is only 5% while it is 7, 23, 45 & 188per cent in India, China, Philippines and U.K., respectively. In India, the difference betweenprice paid by consumers for value added products and farmer's realization has been increasing rapidly.
Time has come when agriculture has to be run as an agribusiness rather than subsistence agriculture. To boost economic return from farming, we must find ways for farmers to earn a greater share of the product sale revenue after adding value to their ownproduce.

Marketing of value added products in more remunerative than selling raw commodities. The demand for agriculture produce has also been changing. With increased income, urbanization and changing eating habits, the demand for processedfood has increased manifold.

Value added agriculture is a process of increasing the economic value andconsumer appeal of an agricultural commodity.

Adding value to grain would probably work best for farmers who are comfortable with doing their own marketing and dealing directly with consumer.

Ways of value-addition to farm produce

There are three ways in which value addition to farm produce is possible.Level 1: (1) Post-harvest level/primary processing - includes proper cleaning,
grading and packaging.

Ex. Vegetables, potatoes, fruit etc. Dehydration of vegetables at surplussupply-fetch more price.

Level 2: (2) Secondary processing - basic processing, packaging and branding.

Ex. Packaed atta, suji, rice etc.

Level 3: (3) High end processing - supply chain management, modern processing technology, packaging of processed foods, branding and marketing. Ex. Potato chips, breakfast food, noodle.

Value added agriculture means getting more income from your farm In innovative ways:
Changing the way a commodity is marketed.
Changing the form of a commodity before it is marketed.
Changing the way a commodity is packaged for market.
Growing a commodity for a special market.
Adding a new enterprise. In many cases, the value added alternatives can becombined to yield an even higher income to the farmer.

Changing the way a commodity is marketed: Add value when you marketa raw agricultural product to command a higher price. Ex. Direct-market your product at a farm stand to special processors or users, to the local community etc. Commodities that require special production methods or harvesting techniques that reach speciality market can be grown under contract for a much higher net return. But you need to seek out these market alternatives.

Changing the form of a commodity before it is marketed: You also addvalue when you transform raw agricultural products through processing. Costs are incurred during processing. Ex. Packing and selling.

Other value – added products include selling flour instead of wheat, or corn meal instead of corn, selling flour directly to the bakery or consumer, sellingvegetables and fruits directly to the consumer than to wholesalers or processors requires cleaning and packaging.

Changing the way a commodity is packaged for market: Value – addedmarketing through packaging provides a great opportunity to increase profit.Package size must meet the consumer's need, if he wants to buy a single tomato, one must not sell only by the basket.

Growing a commodity for a special market: producing specialityagricultural products for export markets.

Adding a new enterprise: A new enterprise is defined as any change in aproduct or service. This includes growing the commodity for a special or niche market.

Ex. A change in production processes, for example might involve switching to organic production practices, it might mean changing corn varieties to produce a special crop for a special industry such as industrial oil. A new enterprise or activity might include adding mushroom, goat production. Weneed to think more broadly about our alternatives who our customers might be. Whichever alternatives you select must be driven by marketing opportunities.

How do value added enterprises contribute to sustainability: Value – addedagriculture –

Sustains the farm by capturing a larger share of the consumer food dollar the direct marketing.
Creates on enterprise that is logical extension of the current farm business.
Provides an innovative business strategy that allows small farms to competewith large farms.
Create new employment opportunities and new markets for high valueagricultural products.
Invigorates the local economy.

Keys to success

Choose something you love to do
Follow demand-driven production
Create a high-quality product
Start small and grow naturally
Make decision based on good records
Establish a loyal customer base, preferably local
Provide more than just food or a product
Get the whole family or all the partners involved
Keep informed - to keep informed about your customers, your competition, the laws concerning your business and other producers.
Plan for the future.

Agriculture Commercialization

Associated with low level of economic growth, Nepal is characterized as a country with a large portion of rural population, high poverty rate and subsistence agriculture. On various levels, all of these factors are interconnected. About eighty percent of the country's population lives in rural areas and agriculture is their primary livelihood where the rural poverty rate is over three times that of urban areas, 35 percent compared to 10 percent (NARC, 2010). This poverty rate can be associated with the subsistence nature of the major means of livelihood in these areas, agriculture. Seventy-eight percent farm holdings have been reported to be producing mainly for home consumption. The proportion of holdings that produce mainly for sale is not even 1 percent, while little over 21% farm families use their farm produce almost equally for both sale and home consumption (CBS, WB, DFID, & ADB, 2006). However, even when such a large portion of the population is into agriculture, being self sufficient on food has also not been a reality for a large section. An estimated 60 percent of households cannot meet their own food needs, especially in mountainous areas, and agricultural production only meets food requirements for three-to-eight months per year (NARC, 2010). Hence, the involvement of the majority of the population in agriculture is very important to analyze in context of social and economic development of Nepal. Agriculture, which employs two third of the country's labor force and contributes to more than one third of Gross Domestic Product (GDP), is the main source of food, income and employment for the majority, especially for the rural population. Hence, agricultural sector is key in issues of economic growth, poverty alleviation, better living standard of the Nepalese people and overall Human Development.

In this context, Commercialization of agriculture has been proposed as a feasible option for economic growth and poverty alleviation. Since the formulation of the Fifth Five-Year Plan (1975–80), agriculture has been the highest priority because economic growth was dependent on both increasing the productivity of existing crops and diversifying the agricultural base for use as industrial inputs (Savada , M. A. , 1991). The adoption of the 20-year Agriculture Perspective Plan (APP) in 1997 reflects the emphasis the government has given on the agricultural sector and its commercialization.

Agricultural commercialization and diversification involve the gradual replacement of integrated farming systems by specialized enterprises for crop, livestock, poultry and aquaculture products. Changes in product mix and input uses are determined largely by the market forces during this transition. Commercialization of agricultural production is an endogenous process and is accompanied by economic growth, urbanization and withdrawal of labor from the agricultural sector.

Rationale of Agriculture Commercialization in Nepal

With a huge percentage of the population in Nepal living under conditions of abject poverty and social deprivation, poverty alleviation is the biggest long-term development challenge for the government. To meet this challenge, Nepal has to focus on achieving a high level of economic growth. Currently, the economy is largely remittance driven and it has been yet another challenge to get the remittance money invested in the productive sector which could escalate the much needed economic growth of Nepal. In this context, agriculture commercialization has been looked upon as an important option in development agendas, both economic and social. While the share of agriculture in total gross domestic product (GDP) has been declining over the years, it still contributes to one-third of the GDP. But then, over the years the overall economic growth rate and the agriculture sector growth have been going downhill. Since agriculture contributes to more than one third of the GDP, this sector not being able to grow as planned has hampered the picture of broader economic growth of Nepal. Overall economic growth rate declined from 4.8 percent in the 1990s to 3.2 percent during 2001-2006. Agriculture virtually stagnated -- agriculture sector growth rate was 2.7 percent per annum in the 90s and 2.8 percent during 2001 to 2006. Marred by low labor productivity, agriculture is not able to contribute to the economy its due (33 percent share of GDP with 66 percent of country’s labor force employed in the sector (Karkee, M., 2008). Hence, this stagnation in the agriculture sector has a huge part in impeding the economic growth of Nepal and fight against poverty and thus commercialization of agriculture has to be extensively discussed.

Opportunities and Challenge for Nepal

Nepal hosts diverse agro-ecological zones and is promising for exports of off-season horticulture, niche products, and non-timber forest products like medicinal plants. Located between

India and China where more than one third of the world population live, there is a huge market.

Nepal’s recent entry into the WTO presents opportunities too.

For the challenges

Challenges such as meeting food safety rules, animal health regulations, and quality standards and other clauses.

Mountainous terrain and poorly developed road network restrict access to markets, constraining agricultural growth and diversification into higher value added and non-farm activities.
Weak and poorly integrated institutions and inadequate technical support for supply chain development have further limited marketing opportunities.

Legal Framework

Agriculture Development Strategy
Agro Business Promotion Policy

Recommendations

Problems of Nepalese Agriculture should be addressed with a three tier approach. The lower-income segment of the population engaged in subsistence agriculture needs to be provided with subsidies, infrastructures for their growth. Another segment of population engaged in agriculture would benefit more through policies aimed at commercialization of agriculture and doesn't need subsidies from the government whereas the third segment of population already engaged in commercial farming would benefit by policies conducive to industrialization of agriculture.
Farmers engaged in the subsistence farming could be helped through private sector, as private sector is willing and able to help them through loans and technology transfers. Nepalese agriculture could be looked upon through the bell curve approach with subsistence level farming at one end, industrialized farming at the other end and commercialized farming at the middle of the curve. Commercialized farming would occupy the majority of the curve.
Water intensive and external input dependent crop agriculture seems untenable for growth and income sustainability in the context of smallholder agriculture. Government should encourage a shift to high value agriculture from the current subsistence oriented agriculture. High value agriculture could take the pressure off the commonly cultivated food grains such as paddy and increase the opportunities for farmers with smaller land holdings through market expansion within and outside the country. It also gives more value to per unit water application and has higher employment elasticity.
Rather than subsidizing the recurring expenses like fertilizers, the government should
Rather than subsidizing the recurring expenses like fertilizers, the government should focus on facilitating the enhancement of technologies used in agriculture. Providing tax concessions on import of tractors and other agricultural machineries would help to increase farmers' access to these technologies and enhance their productivity.
The government should also work towards increasing the role of private sector in the agriculture sector of the economy. Subsidy on fertilizers, which was intended to assist the farmers, has ended up with negative unintended consequences such as unavailability of fertilizers on time and farmers are being forced to inferior quality fertilizers at exorbitant prices in black markets.
The government in partnership with private sector should establish modern, well-equipped collection centers and storage houses with capacities of holding at least 10 thousand metric ton of each major crop in major market areas such as Jhapa, Chitwan, Bhairahawa and Biratnagar which are also close to agricultural areas. Establishment of such collection centers and storage houses would enable farmers to store their products until they get their desired prices in the market. A 'mundi' (local market for agricultural products) should accompany the collections centers so that buyers and sellers can conduct their transactions easily.

By encouraging the remittance incomes in the rural areas to be invested into high value agriculture, the government could not only boost the productivity and living standards of rural farmers, it could also encourage rural youths to be engaged into agriculture. Agricultural enterprise advisory services, enterprise schemes, enterprise management and skills trainings could be instrumental for attracting rural youths (including the back-home migrants) into agriculture.

The recommendations can be summarized in the following points:

Three-tier approach should be adopted to address the problems of Nepalese agriculture.

Mechanisms to ensure the effective participation of Private sector at different levels is imperative to help the farmers engaged in subsistence level farming.

Government should encourage a shift to high value agriculture from the current subsistence oriented agriculture.

Provisions of subsidies in agricultural fertilizers should be eliminated in favor of open access to private sector.

Government should establish collection centers and storage houses in major market areas so that farmers can easily store their products until they get proper prices for their products.

Government should provide tax concessions on import and application of machineries and technologies used in agriculture such as tractors.

The remittance incomes in the rural areas of the country should be directed towards high-value agriculture

Climate Change and Agriculture

Immediate Actions needed:

Reducing greenhouse gases (GHG) emissions to avoid the worst impacts of climate change in the future

Adapting our systems to cope with the unavoidable change

Public policy intervention to promote adaptation in the agriculture sectors

Reducing the vulnerability of those least able to adapt

Provision of information to stimulate widespread adoption of adaptation techniques widespread

Enhanced role for provision of public goods associated with agriculture

Agriculture itself is a major contributor of greenhouse gases – 5.1 to 6.1 giga tones co2-eq/year in 2005 accounting 10-12% of total global emissions of GHG.

Climate Change Projections

Reduction in water availability in already water stressed areas

Changes in the incidence of extreme events

Impact of sea level rise in the coastal area

Emergence of new pathogens and diseases

Modern agriculture initiatives to minimize the impact

Irrigation

Substitution of labor with energy intensive practices

Plant breeding for heat or water stress tolerant crops

Primary effects and Interactions

Effect of elevated CO2

Interaction of elevated CO2 with other factors

Increased frequency of extreme events

Impacts of weeds, pests, diseases and animal health

Interaction with air pollutants

Impacts and sensitivity in Agriculture

Uncertainty Issue

Estimates of global production, trade and food security

Impact in food prices

Adaptation-Two options

There are three roles for public policy intervention to promote adaptation in the agricultural sector, reducing the vulnerability of those least able to adapt, provision of information to stimulate widespread adoption of adaption techniques and opportunities and an enhanced role for provision of public goods associated with agriculture.

Beyond adaption Agriculture itself is a major source of global greenhouse emission, accounting for an estimated 10-12% contributor of total greenhouse gases.

Background

Agriculture is essentially a manmade adjunct to natural ecosystems and is weather and climate dependent. It is also a significant source of anthropogenic emissions of greenhouse gases, which are coming under increasing scrutiny as countries seek to meet binding mitigation targets. New challenges are emerging in terms of how we interpret the impacts of warming, how farming systems adapt or are adapted to these changes and how near term emissions mitigation requirements can take place in ways that are consistent with longer term adaption plans.

Climate factors constitute some of the main constrains on crop and livestock production and till recently have been assumed as exogenous and unchanging. While farming has a history of responding to changing conditions, whether they are economic, social, and political or climate related, the potential increase in frequency and intensity of extreme climatic events, and other challenges posed by climate change, now gives rise to a need to re-appraise the adaptive capacity of agricultural systems. As Governments throughout the world are assessing the diverse threats posed by climate change, the impacts on agriculture have been identified as potentially the most serious in terms of number of people affected and the severity of impacts on those least able to cope. IPPC projected that under past emissions and present commitments to greenhouse emissions, global mean temperature are likely to rise by approximately 1.4 to 5.8 degree Celsius by 2100. Such changes will have direct impacts on crop yields for example through heat stress, and indirect impacts through associated precipitation changes and through related enhanced atmospheric CO_2. The impacts on social risk and production variability are much more complex.

Climate Change Projection

The impacts of climate change are likely to be greater on those countries more dependent on primary sector economic activities, primarily because of the increase in uncertainty on productivity on these primary sectors. Impact include reduction in water availability in already water stressed areas, changes in the incidence of extreme events such as typhoons and droughts and impacts of sea level rise in low lying coastal areas. Modern agriculture has tried to minimize the impacts of climate and weather uncertainty through irrigation, the substitution of labor with energy intensive practices and plant breeding for heat or water stress tolerant crops. Thus adaption in agriculture takes place either by farmers individually, by farmers and local institutions collectively or through national level policy decisions which provide finance, research and development, and knowledge transfer and property rights or legal frameworks to enable individual or collective action.

3.1.6

HYDRAULICS OF ALLUVIAL CHANNELS

GENERAL

Civilization prospered in agricultural lands by the side of rivers. From the beginning of civilization, mankind has given attention to the problems of rivers. The boundaries of many rivers consist of loose material, which may be carried by the water flowing in these rivers. Depending upon the prevailing conditions, the loose material may either get deposited or scoured. Thus, the boundaries of such a river channel are mobile and not rigid. A change in discharge of water flowing in a rigid boundary channel will cause a change only in the depth of flow. But, in case of mobile (or loose) boundary channels, a change in discharge may cause changes in cross-section, slopes, plan-form of the channel, bed forms and roughness coefficient. The application of the theory of rigid boundary channels to loose boundary channels is, therefore, not correct. Evidently, the problem of mobile boundary channels is more complicated.

The bed of a river channel generally consists of non-cohesive sediment (i.e., silt, sand, and gravel) and such rivers are called alluvial rivers.

Sediment (also known as alluvium) is defined as the loose and noncohesive material through which a river or channel flows. Sediment is also defined as fragmental material transported by, suspended in, or deposited by water or air, or accumulated in the beds by other natural agents. Ice, logs of wood, and organic materials flowing with water are excluded from the definition of sediment.

A channel (or river) flowing through sediment and transporting some of it along with the flowing water is called an alluvial channel (or river). The complex nature of alluvial channel problems stands in the way of obtaining analytical solutions, and experimental methods are generally adopted for obtaining solutions of problems to alluvial channels.

INCIPIENT MOTION OF SEDIMENT

Consider the case of flow of clear water in an open channel of a given slope with a movable bed of non-cohesive material. At low discharges, the bed material remains stationary and, hence, the channel can be treated as rigid. With the increase in discharge, a stage will come when the shear force exerted by the flowing water on a particle will just exceed the force opposing the movement of the particle. At this stage, a few particles on the bed move intermittently. This condition is called the incipient motion condition or, simply, the critical condition.

A knowledge of flow at the incipient motion condition is useful in fixing slope or depth for clear water flow in an alluvial channel. Knowledge of the incipient motion condition is also required in some methods of calculation of sediment load. Hence, there is a need to understand the phenomenon which initiates motion of sediment particles.

The experimental data on incipient motion condition have been analysed by different investigators using one of the following three approaches (1):

Competent velocity approach,

Lift force approach, and

Critical tractive force approach.

Competent velocity is the mean velocity of flow which just causes a particle to move. A relationship among the size of the bed material, its relative density, and the competent velocity is generally developed and used.

Investigators using the lift force approach assume that the incipient motion condition is established when the lift force exerted by the flow on a particle just exceeds submerged weight of the particle.

The critical tractive force approach is based on the premise that it is the drag (and not lift) force exerted by the flowing water on the channel bed which is responsible for the motion of the bed particles.

Of these three approaches, the critical tractive force approach is considered most logical and is most often used by hydraulic engineers. Hence, only this approach has been discussed here.

The critical tractive (or shear) stress is the average shear stress acting on the bed of a channel at which the sediment particles just begin to move. Shields (2) was the first investigator to give a semi-theoretical analysis of the problem of incipient motion. According to him, a particle begins to move when the fluid drag F_1 on the particle overcomes the particle resistance F_2. The fluid drag F_1 is given as

$$F_1 = k_1 \left[C_D \, d^2 \, \frac{1}{2} \rho u_d^2 \right]$$

and the particle resistance F_2 is expressed as

$$F_2 = k_2 \left[d^3 (\rho_s - \rho) g \right]$$

where, C_D = the drag coefficient,

d = the size of the particle,

ρ = the mass density of the flowing fluid,

u_d = the velocity of flow at the top of the particle,

ρ_s = the mass density of the particle,

g = acceleration due to gravity,

k_1 = a factor dependent on the shape of the particle, and

k_2 = a factor dependent on the shape of the particle and angle of internal friction.

Using the Karman-Prandtl equation for the velocity distribution, the velocity u_d can be expressed as

$$\frac{u_d}{u_*} = f_1 \left(\frac{u_* d}{\nu} \right) = f_1 (R^*)$$

Here, ν is the kinematic viscosity of the flowing fluid, u_* the shear velocity equal to $\sqrt{\tau_0 / \rho}$ and τ_0 is the shear stress acting on the boundary of the channel.

Similarly,

$$C_D = f \left(\frac{u_d d}{\nu} \right)$$

or

$$C_D = f_2 \left(\frac{u_* d}{\nu} \right) = f_2 (R^*)$$

Thus,

$$F_1 = k_1 f_2 (R^*) \, d^2 \, \frac{1}{2} \rho u_*^2 \, [f_1 (R^*)]^2$$

At the incipient motion condition, the two forces F_1 and F_2 will be equal. Hence,

2

k f (R*) d2 1 □ u   2 [f (R*)]2 □ k [d3 (□ □□) g]

1 2 c 2 *c 1 c 2 s

Here, the subscript c has been used to indicate the critical condition (or the incipient motion condition). The above equation can be rewritten as

□ u2 2 k *

*c =______2 f (Rc)

Alternatively, (□s □□) gd k1

□* = f (R*) (7.1)

c c

where, □* = □ □c

c □ □s gd

□c = □ u2

and □ □s = □s − □ *

On plotting the experimental data collected by different investigators, a unique relationship between □c* and Rc* was obtained by Shields (2) and is as shown in Fig. 7.1. Thecurve shown in the figure is known as the Shields curve for the incipient condition. The

parameter R * = {u*c d} is, obviously, the ratio of the particle size d and □/u . The parameter

c H □ K *c

□ □ is a measure of thickness of laminar sublayer, i.e., □□. Hence, R* can be taken as a measure

u*c c

of the roughness of the boundary surface. The boundary surface is rough at large values of Rc* and, hence, □c* attains a constant value of 0.06 and becomes independent of Rc* at Rc* □ 400. This value of Rc* (i.e., 400), indicating that the boundary has become rough, is much higher than the value of 70 at which the boundary becomes rough from the established criterion

d

□□ > 6.0. Likewise, the constant value of □c* equal to 0.06 is also on the higher side.

Alternatively, one may use the following equation of the Shields' curve for the direct computation of □c (3) :

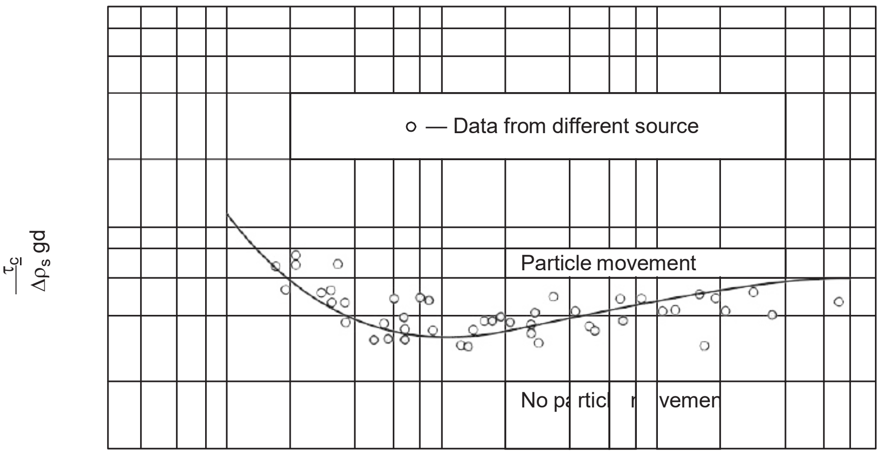
□ □c   0.06 d2

= 0.243 + ______* (7.2)

F □ □2 1/3 (3600 □ d2)1/2 *

in which, d∗ = $\dfrac{d}{(\,\square\,\square^2 / \square\,\square_s\, g)^{1/3}}$

1.0



0.10

0.01 10 R_c^*

0.4 1.0 100 1000

Fig. 7.1 Shields curve for incipient motion condition (2)

For specific case of water (at 20°C) and the sediment of specific gravity 2.65 the above relation for τ_c simply reduces to

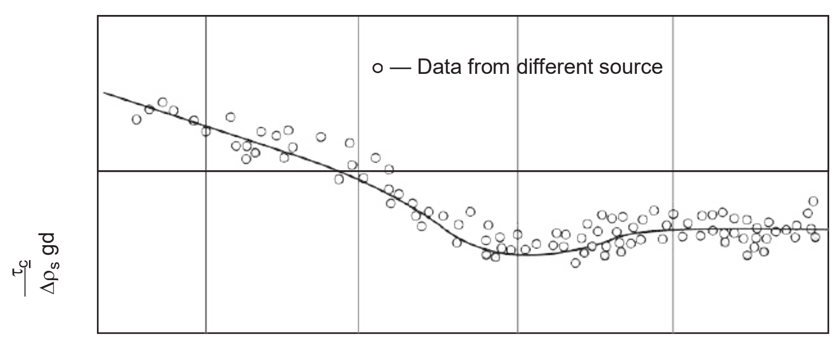
$$\tau_c = 0.155 + \frac{0.409\, d^2}{(1 + 0.177\, d^2)^{1/2}} \qquad (7.3)$$

in which τ_c is in N/m^2 and d is in mm. Equations (7.2) and (7.3) are expected to give the value of τ_c within about ± 5% of the value obtained from the Shields curve (3).

Yalin and Karahan (4) developed a similar relationship (Fig. 7.2) between τ_c^* and R_c^* using a large amount of experimental data collected in recent years. It is noted that at higher values of R_c^* (> 70) the constant value of τ_c^* is 0.045. This relation (Fig. 7.2) is considered better than the more commonly used Shields' relation (1).

10^0

10^{-1}



10

10^{-2} 10^{-1} 0 1

10 10
R_c*

2 3
10 10

Fig. 7.2 Yalin and Karahan curve for incipient motion condition (4)

For given values of d, γ_s, ρ, and ν, the value of τ_c can be obtained from Fig. 7.1 or Fig. 7.2 only by trial as τ_c appears in both parameters τ_c^* and R_c^*. However, the ratio of R_c^* and $\sqrt{\tau_c^*}$ yields a parameter R_0^* which does not contain τ_c and is uniquely related to τ_c^*.

$$R_0^* = \frac{R_c^*}{\sqrt{\tau_c^*}} = \frac{u_{*c}\, d}{\nu \sqrt{\tau_c^*}} = \left(\frac{\Delta\gamma_s\, d}{\rho}\right)^{1/2} \frac{d}{\nu} = \left(\frac{\Delta\rho_s}{\rho}\, g d^3\right)^{1/2} \Big/ \nu \qquad (7.4)$$

Since R_c^* is uniquely related to τ_c^* (Fig. 7.2), another relationship between R_0^* and τ_c^* can be obtained using Fig. 7.2 and Eq. (7.4).
The relationship between R_0^* and τ_c^* is as shown in Fig. 7.3 and can be used to obtain direct solution for τ_c for given values of d, γ_s, ρ and ν.

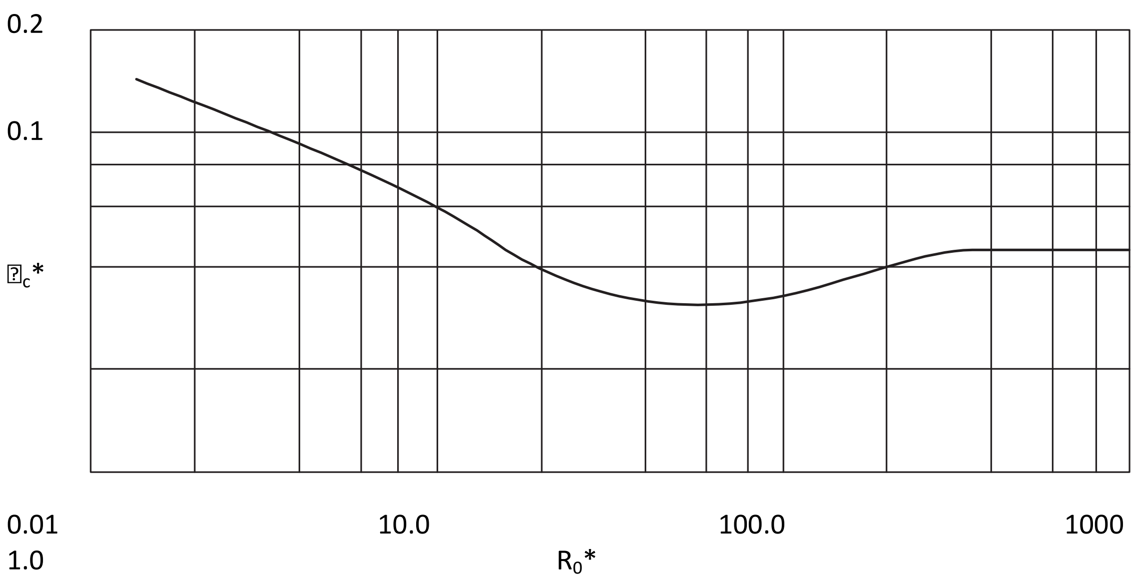


Fig. 7.3 Variation of R_0^* and τ_c^* based on Fig. 7.2

Example 7.1 Water flows at a depth of 0.3 m in a wide stream having a slope of 1×10^{-3}. The median diameter of the sand on the bed is 1.0 mm. Determine whether the grains are stationary or moving ($\nu = 10^{-6}$ m^2/s).

Solution:

$$R_0^* = \left(\frac{\Delta\rho_s}{\rho}\, g d^3\right)^{1/2} \Big/ \nu = \left(1.65 \times 9.81 \times (1 \times 10^{-3})^3\right)^{1/2} \Big/ 10^{-6} = 127.23$$

From Fig. 7.3, $\tau_c^* = 0.035 = \frac{\tau_c}{\Delta\gamma_s d} = \frac{\tau_c}{}$

$c \quad \rho_s \; gd \; 1650 \times 9.81 \times (1.0 \times 10^{-3})$

$\therefore \quad \tau_c = 0.5665 \text{ N/m}^2$

Shear stress on the bed, $\tau_0 = \gamma hS = 9810 \times 0.3 \times 10^{-3} = 2.943 \text{ N/m}^2$

Since $\tau_0 > \tau_c$, the grains would move. τ_c can also be computed from Eq. (7.3).

$$\tau_c = 0.155 + \frac{0.409\,(1.0)^2}{[1 + 0.177\,(1.0)^2]^{1/2}}$$

$= 0.532 \text{ N/m}^2$.

REGIMES OF FLOW

When the average shear stress on the bed of an alluvial channel exceeds the critical shear stress, the bed particles are set in motion and thus disturb the plane bed condition. Depending upon the prevailing flow conditions and other influencing parameters, the bed and the water surface attain different forms. The features that form on the bed of an alluvial channel due to the flow of water are called 'bed forms', 'bed irregularities' or 'sand waves'. Garde and Albertson introduced another term 'regimes of flow' defined in the following manner:

'As the sediment characteristics, the flow characteristics and/or fluid characteristics are changed in alluvial channel, the nature of the bed surface and the water surface changes accordingly. These types of the bed and water surfaces are classified according to their characteristics and are called regimes of flow.'

Regimes of flow will affect considerably the velocity distribution, resistance relations, and the transport of sediment. The regimes of flow can be divided into the following four categories:

Plane bed with no motion of sediment particles,

Ripples and dunes,

Transition, and

Antidunes.

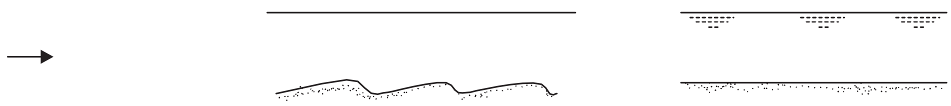
Plane Bed with no Motion of Sediment Particles

When sediment and flow characteristics are such that the average shear stress on the bed is less than the critical shear stress, the sediment particles on the bed do not move. The bed remains plane and the channel boundary can be treated as a rigid boundary. The water surface remains fairly smooth if the Froude number is low. Resistance offered to the flow is on account of the grain roughness only, and Manning's equation can be used for prediction of the mean velocity of flow with Manning's n obtained from the Strickler's equation, as discussed later in this chapter.

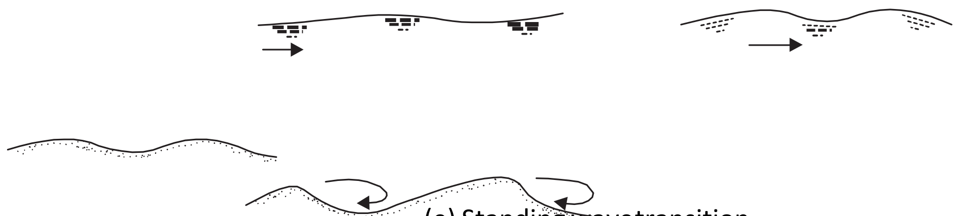
Ripples and Dunes

The sediment particles on the bed start moving when the average shear stress of the flow τ_0 exceeds the critical shear τ_c. As a result of this sediment motion, small triangular undulations known as ripples form on the bed [Fig. 7.4 (a)]. Ripples do not occur if the sediment is coarser than 0.6 mm. The length (between two adjacent troughs or crests) of the ripples is usually less than 0.4 m and the height (trough to crest) does not exceed 40 mm. The sediment motion is confined to the region near the bed and the sediment particles move either by sliding or taking a series of hops.

With the increase in discharge (and, hence, the average shear stress τ_0) the ripples grow into dunes [Fig. 7.4 (b)]. Dunes too are triangular undulations but of larger dimensions. These undulations are also unsymmetrical with a flat upstream face inclined at about 10-20° with the horizontal and steep downstream face whose angle of inclination with the horizontal is approximately equal to the angle of repose of the sediment material. Sometimes, ripples appear on the upstream face of a dune. The dunes in laboratory flumes may have length and height up to about 3 m and 0.4 m, respectively. But, in large rivers, the dunes may be several hundred metres long and up to about 15 m in height. The water surface falls over the crest of dunes and, hence, the water surface waves are out of phase with the bed waves. The flow conditions still correspond to the subcritical range. While most of the sediment particles move along the bed, some finer particles of the sediment may go in suspension.

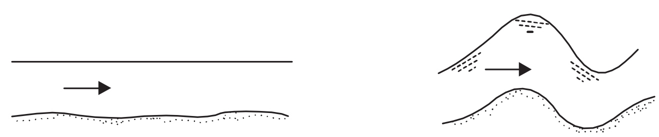


Ripples (d) Plane bed with sediment motion



(e) Standing wave transition

Dunes



Washed out dunes (f) Antidunes

Fig. 7.4 Regimes of flow in alluvial channels

Ripples and dunes have many common features and, hence, are generally dealt with together as one regime of flow. Both ripples and dunes move downstream slowly. Kondap and Garde (6) have given an approximate equation for the advance velocity of ripples and dunes, U_w, as follows:

$$\frac{U_w}{U} = 0.021\left(\frac{U}{\sqrt{gh}}\right)^3 \qquad (7.5)$$

Here, U is the mean velocity of flow, and h is the average depth of flow. The average length (L) and height (H) of ripples and dunes can be estimated from the equations proposed by Ranga Raju and Soni (7):

$$\frac{H}{d}\left(\frac{U}{\sqrt{(\Delta\rho_s/\rho)\,gd}}\right)^3\left(\frac{U}{\sqrt{gR}}\right) = 6500\,(\tau_*')^{8/3} \qquad (7.6)$$

$$\frac{L}{d}\left(\frac{U}{\sqrt{(\Delta\rho_s/\rho)\,gd}}\right)^3\left(\frac{U}{\sqrt{gR}}\right) = 1.8\times10^{8}\,(\tau_*')^{10/3} \qquad (7.7)$$

in which,

$$\tau_*' = \frac{\rho R' S}{\Delta\rho_s d} \qquad (7.8)$$

Here, R' (i.e., hydraulic radius corresponding to the grain roughness) is obtained from the equation,

$$U = \frac{1}{n_s} R'^{2/3} S^{1/2} \quad (7.9)$$

n_s (i.e., Manning's roughness coefficient for the grains alone) is calculated from Strickler's equation,

$$n_s = \frac{d^{1/6}}{25.6} \qquad (7.10)$$

in which d is in metres. R is the hydraulic radius of the channel.

Transition

With further increase in the discharge over the duned bed, the ripples and dunes are washed away, and only some very small undulations are left [Fig. 7.4 (c)]. In some cases, however, the bed becomes plane but the sediment particles are in motion [Fig. 7.4 (d)]. With slight increase in discharge, the bed and water surfaces attain the shape of a sinusoidal wave form. Suchwaves, known as standing waves [Fig. 7.4 (e)], form and disappear and their size does not increase much. Thus, in this regime of transition, there is considerable variation in bed forms from washed out dunes to plane bed with sediment motion and then to standing waves. The Froude number is relatively high. Large amount of sediment particles move in suspension besides the particles moving along the bed. This regime is extremely unstable. The resistance to flow is relatively small.

Antidunes

When the discharge is further increased and flow becomes supercritical (i.e., the Froude numberis greater than unity), the standing waves (i.e., symmetrical bed and water surface waves) move upstream and break intermittently. However, the sediment particles keep on movingdownstream only. Since the direction of movement of bed forms in this regime is opposite to that of the dunes, the regime is termed antidunes, [Fig. 7.4 (f)]. The sediment transport rate is, obviously, very high. The resistance to flow is, however, small compared to that of the ripple and dune regime. In the case of canals and natural streams, antidunes rarely occur.

Importance of Regimes of Flow

In case of rigid boundary channels, the resistance to flow is on account of the surface roughness (i.e., grain roughness) only except at very high Froude numbers when wave resistance may also be present. But, in the case of alluvial channels, the total resistance to flow comprises the form resistance (due to bed forms) and the grain resistance. In the ripple and dune regime, the form resistance may be an appreciable fraction of the total resistance. Because of the varying conditions of the bed of an alluvial channel, the form resistance is a highly varying quantity. Any meaningful resistance relation for alluvial channels shall, therefore, be regime-dependent. It is also evident that the stage-discharge relationship for an alluvial channel will also be affected by regimes of flow.

The form resistance, which is on account of the difference in pressures on the upstream and downstream side of the undulations, acts normal to the surface of the undulations. As such, the form resistance is rather ineffective in the transport of sediment. Only grain shear (i.e., the shear stress corresponding to grain resistance) affects the movement of sediment.

Prediction of Regimes of Flow

There are several methods for the prediction of regimes. The method described here has been proposed by Garde and Ranga Raju (8).

The functional relationship for resistance of flow in alluvial channels was written, following the principles of dimensional analysis, as follows:

$$\frac{U}{\sqrt{(\Delta\rho_s/\rho)\,gR}} = f\left(\frac{R}{d}, S, \frac{g^{1/2}d^{3/2}}{\nu}, P\right) \tag{7.11}$$

Here, S is the slope of the channel bed. Since resistance to flow and the regime of flow are closely related with each other, it was assumed that the parameters on the right-hand side of Eq. (7.11) would predict the regime of flow. The third parameter (i.e., $g^{1/2} d^{3/2}/\nu$) was dropped from the analysis on the plea that the influence of viscosity in the formation of bed waves is rather small. The data from natural streams, canals, and laboratory flumes in which the regimes had also been observed, were used to develop Fig. 7.5 on which lines demarcating the regimes of flow have been drawn. The data used in developing Fig. 7.5 cover a wide range of depth offlow, slope, sediment size, and the density of sediment.

It should be noted that the lines of 45° slope on Fig. 7.5 – such as the line demarcating ‘no motion’ and ‘ripples and dunes’ regimes – represent a line of constant value of $\tau_* = \frac{\rho RS}{\Delta\rho_s d}$.

This means that different regimes of flow can be obtained at the same shear stress by varying suitably the individual values of R and S. Therefore, shear stress by itself cannot adequately define regimes of flow.

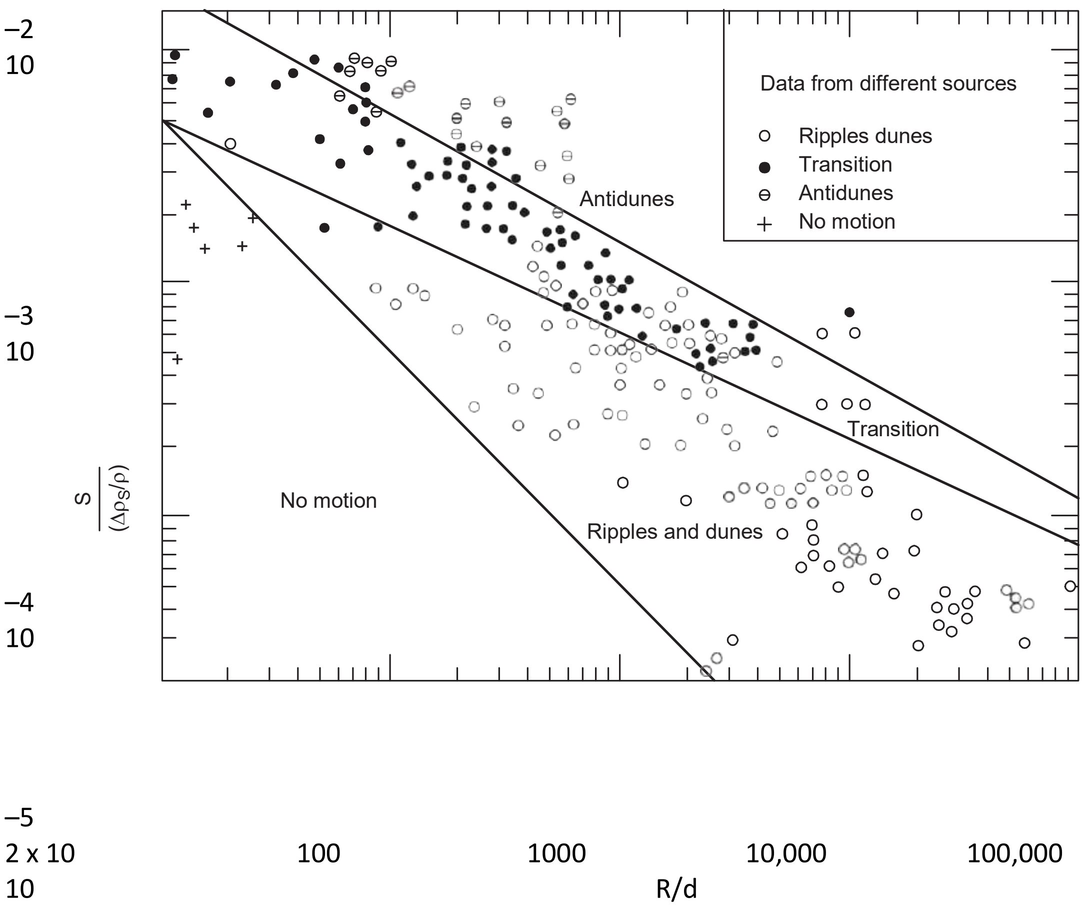


Fig. 7.5 Predictor for regimes of flow in alluvial channels (8)

The method of using Fig. 7.5 for prediction of regimes of flow consists of simply calculating the parameters R/d and $S/(\Delta\rho_s/\rho)$ and then finding the region in which the corresponding point falls. One obvious advantage of this method is that it does not require knowledge of the mean velocity U and is, therefore, suitable for prediction of regimes for resistance problems.

Example 7.2 An irrigation canal has been designed to have R = 2.5 m and S = 1.6 × 10^{-4}. The sediment on the bed has a median size of 0.30 mm. Find: (i) the bed condition that may be expected, (ii) the height and spacing of undulations, and (iii) the advance velocity of the undulations. Assume depth of flow and mean velocity of flow to be 2.8 m and 0.95 m/s, respectively.

Solution:

$$\frac{R}{d} = \frac{2.5}{0.3 \times 10^{-3}} = 8333.33$$

$$\frac{S}{\Delta\rho_s/\rho} = \frac{1.6 \times 10^{-4}}{1.65} = 9.7 \times 10^{-5}$$

From Fig. 7.5, the expected bed condition would correspond to 'ripples and dunes' regime. From Eq. (7.10),

$$n_s = \frac{d^{1/6}}{25.6} = \frac{(0.3 \times 10^{-3})^{1/6}}{25.6} = 0.01$$

and from the Manning's equation [Eq. (7.9)],

$$U = \frac{1}{n_s} R'^{2/3} S^{1/2}$$

$$\therefore \quad R' = \left(\frac{U n_s}{S^{1/2}}\right)^{3/2} = \left(\frac{0.95 \times 0.01}{(1.6 \times 10^{-4})^{1/2}}\right)^{3/2} = 0.651 \text{ m}$$

$$\tau_*' = \frac{R'S}{(\Delta\rho_s/\rho)d} = \frac{0.651 \times 1.6 \times 10^{-4}}{1.65 \times 0.3 \times 10^{-3}} = 0.21$$

Using Eq. (7.6),

$$\frac{H}{d}\left(\frac{U}{\sqrt{gR}}\right)^3 \frac{U}{\sqrt{(\Delta\rho_s/\rho)gd}} = 6500\,(\tau_*')^{8/3}$$

$$\frac{H}{0.3 \times 10^{-3}}\left(\frac{0.95}{\sqrt{9.81 \times 2.5}}\right)^3 \frac{0.95}{\sqrt{9.81 \times 1.65 \times 0.3 \times 10^{-3}}} = 6500\,(0.21)^{8/3}$$

$\therefore$ H = 0.316 m

Similarly, from Eq. (7.7)

$$\left(\frac{U}{\sqrt{gR}}\right)^3 \frac{U}{\sqrt{(\Delta\rho_s/\rho)gd}} \quad \frac{L}{d}$$

$$= 1.8 \times 10^8 \, (\;*\;)^{10/3}$$

i.e., $\left(\dfrac{0.95}{\sqrt{9.81 \times 1.65 \times 0.3 \times 10^{-3}}}\right)^3 \quad \dfrac{0.95}{} \quad \dfrac{L}{\sqrt{9.81 \times 0.25 \times 10^{-3}}} \quad = 1.8 \times 10^8 \, (0.21)^{10/3}$

$\therefore \quad L = 0.611$ m

$\dfrac{U_w}{U} = 0.021 \quad \left(U\right)^3 \qquad \left(\dfrac{0.95}{\sqrt{9.81 \times 2.8}}\right)^3$

$\therefore \quad U_W = 0.95 \times 0.021 \quad = 0.43$ m/hr

RESISTANCE TO FLOW IN ALLUVIAL CHANNELS

The resistance equation expresses relationship among the mean velocity of flow U, the hydraulicradius R, and the characteristics of the channel boundary. For steady and uniform flow in rigid boundary channels, the Keulegan's equations (logarithmic type) or power-law type of equations (like the Chezy's and the Manning's equations) are used. Keulegan (9) obtained thefollowing logarithmic relations for rigid boundary channels:

For smooth boundaries,

$$\frac{U}{u_*} = 5.75 \log \left(\frac{u_* R}{\nu}\right) + 3.25 \qquad (7.12)$$

For rough boundaries,

$$\frac{U}{u_*} = 5.75 \log (R/k_s) + 6.25 \qquad (7.13)$$

For the range $5 < \frac{R}{k_s} < 700$, the Manning's equation,

$$U = \frac{1}{n} R^{2/3} S^{1/2} \qquad (7.14)$$

has been found (9) to be as satisfactory as the Keulegan's equation [Eq. (7.13)] for rough boundaries. In Eq. (7.14), n is the Manning's roughness coefficient which can be calculated using the Strickler's equation,

$$n = \frac{k_s^{1/6}}{25.6} \qquad (7.15)$$

Here, k_s is the equivalent sand grain roughness in metres. Another power-law type of equationis given by Chezy in the following form:

$$U = C\sqrt{RS} \qquad (7.16)$$

Comparing the Manning's equations,

$$\frac{U}{u_*} = \frac{C}{\sqrt{g}} = \frac{R^{1/6}}{n\sqrt{g}} = \frac{25.6\left(\frac{R}{k_s}\right)^{1/6}}{\sqrt{g}} \qquad (7.17)$$

In case of an alluvial channel, so long as the average shear stress τ_0 on boundary of the channel is less than the critical shear τ_c, the channel boundary can be considered rigid and any of the resistance equations valid for rigid boundary channels would yield results for alluvial channels too. However, as soon as sediment movement starts, undulations develop on the bed, thereby increasing the boundary resistance. Besides, some energy is required to move the grains. Further, the sediment particles in suspension also affect the resistance of alluvial streams. The suspended sediment particles dampen the turbulence or interfere with the production of turbulence near the bed where the concentration of these particles as well as the rate of turbulence production are maximum. It is, therefore, obvious that the problem of resistance in alluvial channels is very complex and the complexity further increases if one includes the effects of channel shape, non-uniformity of sediment size, discharge variation, and other factors on channel resistance. None of the resistance equations developed so far takes all these factors into consideration.

The method for computing resistance in alluvial channels can be grouped into two broad categories. The first includes such methods which deal with the overall resistance and use either a logarithmic type relation or a power-law type relation for the mean velocity. The second category of methods separates the total resistance into grain resistance and form resistance (i.e., the resistance that develops on account of undulations on the channel bed). Both categories of methods generally deal with uniform steady flow.

Resistance Relationships based on Total Resistance Approach

The following equation, proposed by Lacey (10) on the basis of analysis of stable channel datafrom India, is the simplest relationship for alluvial channels:

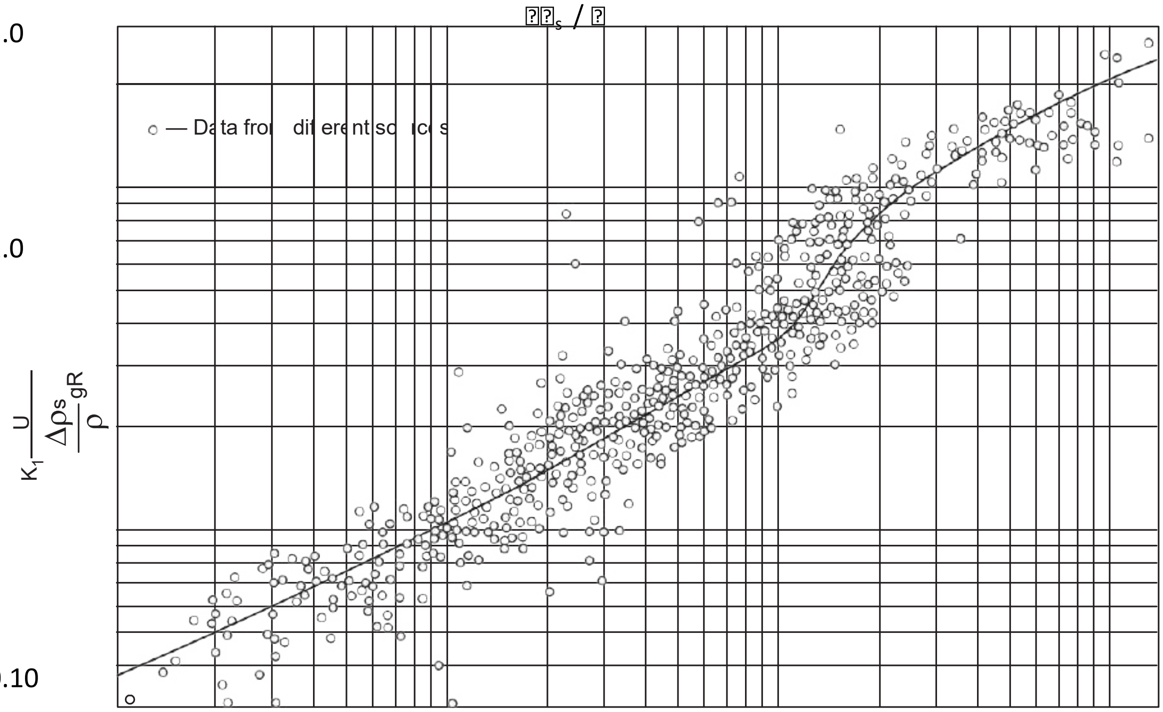
$$U = 10.8R^{2/3} S^{1/3} \qquad (7.18)$$

However, this equation is applicable only under regime conditions (see Art. 8.5) and, hence, has only limited application.

Garde and Ranga Raju (11) analysed data from streams, canals, and laboratory flumes to obtain an empirical relation for prediction of mean velocity in an alluvial channel. The functional relation, [Eq. (7.11)] may be rewritten (11) as

$$\frac{U}{\sqrt{(\Delta\rho_s / \rho)\, g R}} = f\left(\frac{R}{d}, \frac{S}{\Delta\rho_s/\rho}, \frac{g^{1/2} d^{3/2}}{\nu}\right) \qquad (7.19)$$

By employing usual graphical techniques and using alluvial channel data of canals, rivers, and laboratory flumes, covering a large range of d and depth of flow, a graphical relation between $K_1 \dfrac{U}{\sqrt{(\Delta\rho_s / \rho)\, gR}}$ and $K_2 \left(\dfrac{R}{d}\right)^{1/3} \dfrac{S}{\Delta\rho_s / \rho}$, Fig. 7.6, was obtained for the prediction of



−3 −2 −1

10 $\underline{R}^{1/3}$ $\underline{S}$ 10 10

$K_2(d)$ (/)

s

Fig. 7.6 Resistance relationship for alluvial channels (12) the mean velocity U. The coefficients K_1 and K_2 were related to the sediment size d by the graphical relations shown in Fig. 7.7. It should be noted that the dimensionless parameter $g^{1/2} d^{3/2}/\nu$ has been replaced by the sediment size alone on the plea that the viscosity of the liquid for a majority of the data used in the analysis did not change much (12). This method is expected to yield results with an accuracy of ± 30 per cent (13). For given S, d, $\Delta\rho_s$, ρ, and the stage-hydraulic radius curve and stage-area curve of cross-section, the stage-discharge curve for an alluvial channel can be computed as follows:

Assume a stage and find hydraulic radius R and area of cross-section A from stage-hydraulic radius and stage-area curves, respectively.

Determine K_1 and K_2 for known value of d using Fig. 7.7.

Compute $K_2 (R/d)^{1/3} \frac{S}{\Delta\rho_s / \rho}$ and read the value of $K_1 \frac{U}{\sqrt{(\Delta\rho_s / \rho)\, g R}}$ from Fig. 7.6.

Calculate the value of the mean velocity U and, hence, the discharge.

Repeat the above steps for other values of stage.

Finally, a graphical relation between stage and discharge can be prepared.

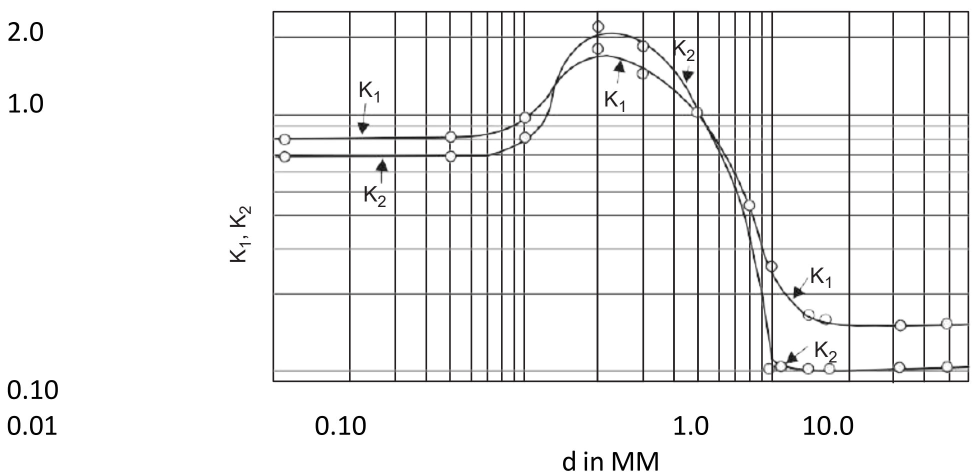


Fig. 7.7 Variation of K_1 and K_2 with sediment size (12)

Example 7.3 An alluvial stream (d = 0.60 mm) has a bed slope of 3×10^{-4} Find the mean velocity of flow when the hydraulic radius is 1.40 m.

Solution:

From Fig. 7.7, $K_1 = 0.75$ and $K_2 = 0.70$

$$K_2 (R/d)^{1/3} \frac{S}{\Delta\rho_s / \rho} = 0.70 \left(\frac{1.40}{0.6 \times 10^{-3}}\right)^{1/3} \frac{3 \times 10^{-4}}{1.65} = 1.69 \times 10^{-3}$$

From Fig. 7.6,

$$K_1 \frac{U}{\sqrt{(\Delta\rho_s / \rho)\, g R}} = 0.135 \quad \text{or} \quad 0.75 \times \frac{U}{\sqrt{1.65 \times 9.81 \times 1.4}} = 0.135$$

$\therefore$ $U = 0.86$ m/s.

7.4.2. Resistance Relationship Based on Division of Resistance

In dealing with open channel flows, hydraulic radius R of the flow cross-section is taken as the characteristic depth parameter. The use of this parameter requires that the roughness over the whole wetted perimeter is the same. Such a condition can be expected in a very wide channel with alluvial bed and banks. However, laboratory flumes with glass walls and sand bed would have different roughnesses on the bed and side walls. In such cases, therefore, the hydraulic radius of the bed R_b is used instead of R in the resistance relations. The hydraulic radius of the bed R_b can be computed using Einstein's method (14) which assumes that the velocity is uniformly distributed over the whole cross-section. Assuming that the total area of cross-section of flow A can be divided into areas A_b and A_w corresponding to the bed and walls, respectively, one can write

$$A = A_w + A_b$$

For rectangular channels, one can, therefore, write

$$(B + 2h)\,R = 2\,hR_w + B\,R_b$$

$$\therefore \quad R_b = \left(1 + \frac{2h}{B}\right) R - \frac{2h}{B} R_w$$

$$= (B + 2h)(R/B) - 2hR_w/B$$

$$= (PR/B) - 2hR_w/B$$

$$= (A/B) - 2hR_w/B$$

$$= h - 2hR_w/B \quad (7.20)$$

Using Manning's equation for the walls, i.e.,

$$U = \frac{1}{n_w} R_w^{2/3} S^{1/2} \quad (7.21)$$

one can calculate the hydraulic radius of the wall R_w if the Manning's coefficient for the walls, n_w is known. Using Eq. (7.20), the hydraulic radius of the bed R_b can be computed.

Example 7.4 A 0.40m wide laboratory flume with glass walls (n_w = 0.01) and mobile bed of 2.0 mm particles carries a discharge of 0.1 m^3/s at a depth of 0.30m. The bed slope is 3×10^{-3}. Determine whether the particles would move or not. Neglect viscous effects.

Solution:

Hydraulic radius, $R = \dfrac{0.3 \times 0.4}{0.4 + 2 \times 0.3} = 0.12$ m

Mean velocity of flow in the flume $= \dfrac{0.1}{0.3 \times 0.4} = 0.833$ m/s

Using Eq. (7.21), $R_w = \left(\dfrac{U n_w}{S^{1/2}}\right)^{3/2}$

$$= \left(\frac{0.833 \times 0.01}{(3 \times 10^{-3})^{1/2}} \right)^{3/2}$$

$$= 0.0593 \text{ m}$$

Using Eq. (7.20), $R_b = 0.3 - \frac{2 \times 0.3}{0.4} \times 0.0593$

$= 0.211$ m

$\therefore$ Bed shear, $\tau_b = \rho g R_b S$

$= 9810 \times 0.211 \times 3 \times 10^{-3}$

$= 6.21\ \text{N/m}^2$

On neglecting viscous effects and using Yalin and Karahan's curve,

$$\frac{\tau_c}{\Delta\rho_s g d} = 0.045$$

$\therefore$ Critical shear, $\tau_c = 0.045 \times 1.65 \times 9810 \times 2 \times 10^{-3}$

$= 1.457\ \text{N/m}^2$

Since $\tau_b > \tau_c$, the particles would move.

Einstein and Barbarossa (15) obtained a rational solution to the problem of resistance in alluvial channels by dividing the total bed resistance (or shear) τ_{ob} into resistance (or shear) due to sand grains τ'_{ob} and resistance (or shear) due to the bed forms τ''_{ob}, i.e.,

$$\tau_{ob} = \tau'_{ob} + \tau''_{ob} \qquad (7.22)$$

or $\rho g R_b S = \rho g R'_b S + \rho g R''_b S$

i.e., $R_b = R'_b + R''_b$ (7.23) where R'_b and R''_b are hydraulic radii of the bed corresponding to grain and form resistances (or roughnesses).

For a hydrodynamically rough plane boundary, the Manning's roughness coefficient for the grain roughness n_s is given by the Strickler's equation i.e.,

$$n_s = \frac{d_{65}^{1/6}}{24.0} \qquad (7.24)$$

Here, d_{65} (in metres) represents the sieve diameter through which 65 per cent of the sediment will pass through, i.e., 65 per cent of the sediment is finer than d_{65}. Therefore, Manning's equation can be written as

$$U = \frac{1}{n_s} R_b'^{2/3} S^{1/2}$$

$$U = \frac{24}{d_{65}^{1/6}} R_b'^{2/3} S^{1/2} \qquad (7.25)$$

Since $U'_* = \sqrt{\tau'_{ob}/\rho} = \sqrt{g R'_b S}$

$$\frac{U}{U'_*} = \frac{24}{d_{65}^{1/6}} \frac{R_b'^{2/3} S^{1/2}}{\sqrt{g R'_b S}}$$

or $\frac{U}{U'_*}$

$$= 7.66 \left(\frac{R_b}{d_{65}}\right)^{1/6} \qquad (7.26)$$

Einstein and Barbarossa (15) replaced this equation with the following logarithmic relation having theoretical support.

$$\frac{U}{U_*'} = 5.75 \log \frac{12.27\, R_b'}{d_{65}} \qquad (7.27)$$

Equation (7.27) is valid for a hydrodynamically rough boundary. A viscous correction factor x (which is dependent on d_{65}/δ', Table 7.1, Fig. 7.8) was introduced in this equation to make it applicable to boundaries consisting of finer material ($d_{65}/\delta' < 10$). The modified equation is (15)

$$\frac{U}{U_*'} = 5.75 \log \frac{12.27\, R_b'\, x}{d_{65}} \qquad (7.28)$$

Table 7.1 Variation of x with d_{65}/δ' (15)

d_{65}/δ'	0.2	0.3	0.5	0.7	1.0	2.0	4.0	6.0	10
x	0.7	1.0	1.38	1.56	1.61	1.38	1.10	1.03	1.0

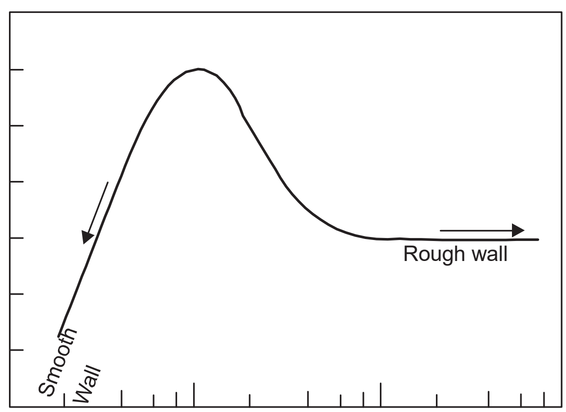


Fig. 7.8 Correction x in Eq. (7.28) (15)

Einstein and Barbarossa (15) recommended that one of the equations, Eq. (7.26) or Eq. (7.27) may be used for practical problems. The resistance (or shear) due to bed forms τ_{ob}'' is computed by considering that there are N undulations of cross-sectional area a in a length of channel L with total wetted perimeter P. Total form drag F on these undulation is given by

$$F = C_D\, a \left(\frac{1}{2}\rho U^2\right) N \qquad (7.29)$$

Here, C_D is the average drag coefficient of the undulations. Since this drag force acts on area LP, the average shear stress τ_{ob}'' will be given as

$$\tau_{ob} = \frac{F}{LP} = \frac{\rho}{LP}\,\frac{C_D a N U^2}{2}$$

$$u_{*ob} = \sqrt{\frac{\tau_{ob}}{\rho}} = U\sqrt{\frac{C_D a N}{2LP}}$$

or

$$\frac{U}{u_*} = \sqrt{2LP/(C_D a N)} \tag{7.30}$$

Here, U_*'' is the shear velocity corresponding to bed undulations. According to Einstein and Barbarossa, the parameters on the right hand side of Eq. (7.30) would primarily depend on sediment transport rate which is a function of Einstein's parameter $\psi' = \Delta_s d_{35}/R'_b S$.

Therefore, they obtained an empirical relation, Fig. 7.9., between $\frac{U}{U_*''}$ and ψ' using field data natural streams. The relationship proposed by Einstein and Barbarossa can be used to compute mean velocity of flow for a given stage (i.e., depth of flow) of the river and also to prepare stage– discharge relationship.

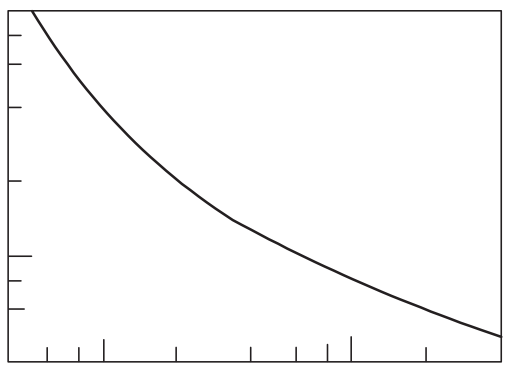


Fig. 7.9 Einstein and Barbarossa relation between U/U_*'' and ψ' (15)

The computation of mean velocity of flow for a given stage requires a trial procedure. From the known channel characteristics, the hydraulic radius R of the flow area can be determined for a given stage (or depth of flow) of the river for which the mean velocity of flowis to be predicted. For a wide alluvial river, this hydraulic radius R approximately equals R_b. A value of R'_b smaller than R_b is assumed and a trial value of the mean velocity U is calculated from Eq. (7.25) or Eq. (7.26) or Eq. (7.27). The value of $\frac{U}{U_*''}$ is read from Fig. 7.9 for ψ' corresponding to the assumed value of R'_b. From known values of U (trial value) and U/U_*'', U_*'' and, hence, R''_b can be computed. If the sum of R'_b and R''_b equals R_b the assumed value of R'_b and, hence, the corresponding mean velocity of flow U computed from Eq. (7.25) or Eq. (7.26) or Eq. (7.27) are okay. Otherwise, repeat the procedure for another trial value of R_b' till the sum of R_b' and R_b'' equals R_b. The computations can be carried out easily in a tabular form as illustrated in the following example:

Example 7.5 Solve Example 7.3 using Einstein and Barbarossa method.

Solution: For given d = 0.6 mm and bed slopes $S = 3 \times 10^{-4}$

$$U_*' = \sqrt{gR_b'S} = \sqrt{9.81 \times R_b' \times 3 \times 10^{-4}} = 0.054\sqrt{R_b'}$$

From Eq. (7.25)

$$U = 7.66\, U_*'(R_b'/d)^{1/6} = 7.66 \times 0.054\sqrt{R_b'}\left(\frac{R_b'}{0.6 \times 10^{-3}}\right)^{1/6}$$

$$\therefore \quad U = 1.4243\, R_b'^{\,2/3}$$

$$\psi' = \frac{\Delta_s d_{35}}{\rho R'_b S} = \frac{1.65 \times 0.6 \times 10^{-3}}{R'_b (3 \times 10^{-4})} = \frac{3.3}{R'_b}$$

From $U''_* = \sqrt{g R''_b S}$

$$R''_b = \frac{(U''_*)^2}{gS} = \frac{(U''_*)^2}{9.81 \times 3 \times 10^{-4}} = 339.79\,(U''_*)^2$$

The trial procedure for computation of mean velocity can now be carried out in a tabular form. It is assumed that the alluvial river is wide and, therefore,

$R_b \approx R$

Trial No.	R'_b (m)	U'_* (m/s)	U (m/s)	ψ'	U/U''_*	U''_* (m/s)	R''_b (m)	R_b (m)	Comments
1	1.2	0.059	1.6084	2.75	16.0	0.1005	3.432	4.632	higher than 1.4
2	0.5	0.038	0.8973	6.60	10.5	0.0855	2.484	2.984	higher than 1.4
3	0.2	0.024	0.4871	16.50	7.0	0.0696	1.646	1.846	higher than 1.4
4	0.1	0.017	0.3069	33.00	5.0	0.0614	1.281	1.381	close to 1.4
5	0.11	0.018	0.3270	30.00	5.2	0.0629	1.344	1.444	higher than 1.4
6	0.105	0.175	0.3170	31.43	5.1	0.0622	1.315	1.420	close to 1.4

Values of R_b ($\approx$ R) in row nos. 4 and 6 are reasonably close to the given value of 1.4 m. Thus, the velocity of flow is taken as the average of 0.3069 m/s and 0.3170 m/s i.e., 0.312 m/s.

The difference in the value of mean velocity obtained by Einstein and Barbarossa method compared with that obtained by Garde and Ranga Raju method (Example 7.3) should be noted.

For preparing a stage-discharge curve, one needs to obtain discharges corresponding to different stages of the river. If one neglects bank friction (i.e., $R = R_b$), the procedure, requiring no trial, is as follows:

For an assumed value of R'_b, the mean velocity of flow U is computed from Eq. (7.26) and U/U''_* is read from Fig. 7.9 for ψ' corresponding to the assumed value of R'_b. From known values of U and U/U''_* one can determine U''_* and, hence, R''_b. The sum of R'_b and R''_b gives R_b which equals R (if bank friction is neglected). Corresponding to this value of R, one can determine the stage and, hence, the area of flow cross-section A. The product of U and A gives the discharge, Q corresponding to the stage. Likewise, for another value of R'_b, one can determine stage and the corresponding discharge.

TRANSPORT OF SEDIMENT

When the average shear stress τ_o on the bed of an alluvial channel exceeds the critical shear τ_c, the sediment particles start moving in different ways depending on the flow condition, sediment size, fluid and sediment densities, and the channel condition.

At relatively low shear stresses, the particles roll or slide along the bed. The particles remain in continuous contact with the bed and the movement is generally discontinuous. Sediment material transported in this manner is termed contact load.

On increasing the shear stress, some sediment particles lose contact with the bed for some time, and ‘hop’ or ‘bounce’. The sediment particles moving in this manner fall into the category of saltation load. This mode of transport is significant only in case of noncohesive materials of relatively high fall velocities such as sand in air and, to a lesser extent, gravel in water.

Since saltation load is insignificant in case of flow of water and also because it is difficult to distinguish between saltation load and contact load, the two are grouped together and termed bed load, which is transported on or near the bed.

With further increase in the shear stress, the particles may go in suspension and remain so due to the turbulent fluctuations. The particles in suspension move downstream. Such sediment material is included in the suspended load. Sediment particles move in suspension when $u_*/w_0 > 0.5$. Here, w_0 is the fall velocity for sediment particles of given size.

The material for bed load as well as a part of the suspended load originates from the bed of the channel and, hence, both are grouped together and termed bed-material load.

Analysis of suspended load data from rivers and canals has shown that the suspended load comprises the sediment particles originating from the bed and the sediment particles which are not available in the bed. The former is the bed-material load in suspension and the latter is the product of erosion in the catchment and is appropriately called wash load. The wash load, having entered the stream, is unlikely to deposit unless the velocity (or the shear stress) is greatly reduced or the concentration of such fine sediments is very high. The transport rate of wash load is related to the availability of fine material in the catchment and its erodibility and is, normally, independent of the hydraulic characteristics of the stream. As such, it is not easy to make an estimate of wash load.

When the bed-material load in suspension is added to the bed-material load moving as bed load, one gets the total bed-material load which may be a major or minor fraction of the total load comprising bed-material load and wash load of the stream depending on the catchment characteristics.

Irrigation channels carrying silt-laden water and flowing through alluvial bed are designed to carry certain amounts of water and sediment discharges. This means that the total sediment load transport will affect the design of an alluvial channel. Similarly, problems related to reservoir sedimentation, aggradation, degradation, etc. can be solved only if the total sediment load being transported by river (or channel) is known. One obvious method of estimation of total load is to determine bed load, suspended load, and wash load individually and then add these together. The wash load is usually carried without being deposited and is also not easy to estimate. This load is, therefore, ignored while analysing channel stability.

It should, however, be noted that the available methods of computation of bed-material load are such that errors of the order of one magnitude are not uncommon. If the bed-material load is only a small fraction of the total load, the foregoing likely error would considerably reduce the validity of the computations. This aspect of sediment load computations must always be kept in mind while evaluating the result of the computations.

Bed Load

The prediction of the bed load transport is not an easy task because it is interrelated with the resistance to flow which, in turn, is dependent on flow regime. Nevertheless, several attempts have been made to propose methods – empirical as well as semi-theoretical – for the computation of bed load. The most commonly used empirical relation is given by Meyer-Peter and Müller (16). Their relation is based on: (i) the division of total shear into grain shear and form shear, and (ii) the premise that the bed load transport is a function of only the grain shear. Their equation, written in dimensionless form, is as follows:

$$\left(\frac{n_s}{n}\right)^{3/2}\frac{\rho RS}{\Delta\rho_s d_a} = 0.047 + 0.25\left(\frac{q_B}{\rho_s g^{3/2} d_a^{3/2}}\right)^{2/3}\left(\frac{\rho}{\Delta\rho_s}\right)^{1/3} \quad (7.31)$$

which is rewritten as

$$\tau_*' = 0.047 + 0.25\,\Phi_B^{2/3} \quad (7.32)$$

or $$\Phi_B = 8\,(\tau_*' - 0.047)^{3/2} \quad (7.33)$$

where, Φ_B is the bed load function and equals $\dfrac{q_B}{\rho_s g^{3/2} d_a^{3/2}\sqrt{\Delta\rho_s/\rho}}$,

τ_*' is the dimensionless grain shear and equals $\dfrac{\tau_o'}{\Delta\rho_s g d_a}$,

and τ_o' is the grain shear and equals $\left(\dfrac{n_s}{n}\right)^{3/2}\rho gRS$.

Here, q_B is the rate of bed load transport in weight per unit width, i.e., N/m/s and d_a is the arithmetic mean size of the sediment particles which generally varies between d_{50} and d_{60}(1).

From Eq. (7.33), it may be seen that the value of the dimensionless shear τ_*' at the incipient motion condition (i.e., when q_B and, hence, Φ_B is zero) is 0.047. Thus, ($\tau_*' - 0.047$) can be interpreted as the effective shear stress causing bed load movement.

The layer in which the bed load moves is called the bed layer and its thickness is generally taken as 2d.

Example 7.6 Determine the amount of bed load in Example 7.2

Solution:

From the solution of Example 7.2,

$\tau_*' = 0.21$

From Eq. (7.33),

$\therefore \quad \Phi_B = 8 \times (0.21 - 0.047)^{3/2} = 0.5265$

i.e., $$\frac{q_B}{\rho_s g^{3/2} d^{3/2}\sqrt{\Delta\rho_s/\rho}} = 0.5265$$

$\therefore \quad q_B = 0.5265 \times 2650 \times (9.81 \times 0.3 \times 10^{-3})^{1.5}\,(1.65)^{1/2}$ N/m/s

= 0.286 N/m/s

A semi-theoretical analysis of the problem of the bed load transport was first attempted by Einstien (14) in 1942 when he did not consider the effect of bed forms on bed load transport. Later, he presented a modified solution (17) to the problem of bed load transport. Einstein's solution does not use the concept of critical tractive stress but, instead, is based on the assumption that a sediment particle resting on the bed is set in motion when the instantaneous hydrodynamic lift force exceeds the submerged weight of the particle. Based on his semi- theoretical analysis, a curve, Fig. 7.9, between the Einstein's bed load parameter

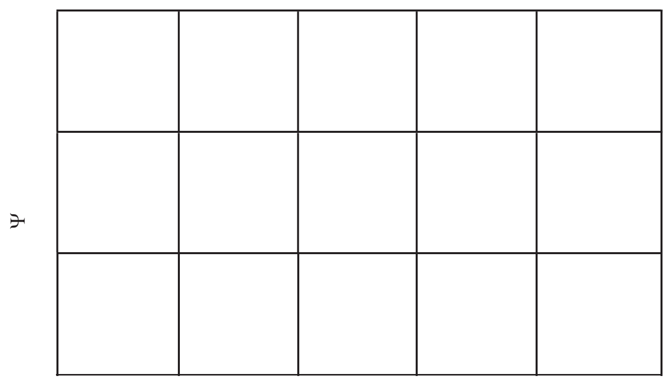
$$\Phi_B \left(= \frac{q_B}{\gamma_s g^{3/2} d^{3/2} \sqrt{\Delta\rho_s / \rho}} \right) \quad \text{and} \quad \Psi \left(= \Delta\gamma_s d / \gamma R' S \right)$$

can be used to compute the bed load transport in case of uniform sediment. The coordinates of the curve of Fig. 7.10 are given in Table 7.2. The method involves computation of Ψ for given sediment characteristics and flow conditions and reading the corresponding value of Φ_B from Fig. 7.10 to obtain the value of q_B.

100

10

1.0



0.1

0.0001 0.001 0.01 0.1 1.0 10

Φ_B

Fig. 7.10 Einstein's bed load transport relation (17)

Table 7.2 Relationship between Φ_B and Ψ (17)

Ψ	27.0	24.0	22.4	18.4	16.4	11.5	9.5	5.5	4.08	1.4	0.70
Φ_B	10^{-4}	5×10^{-4}	10^{-3}	5×10^{-3}	10^{-2}	5×10^{-2}	10^{-1}	5×10^{-1}	1.0	5.0	10.0

Example 7.7 Determine the amount of bed load in Example 7.2 using Einstein's method.

Solution: From the solution of Example 7.2,

$R' = 0.651$ m

$$\therefore \quad \Psi = \frac{\Delta\gamma_s d}{\gamma R' S}$$

$$= \frac{1.65 \times 0.3 \times 10^{-3}}{0.651 \times 1.6 \times 10^{-4}}$$

$= 4.752$

$\therefore \quad \Phi_B = 0.763$ (from Fig. 7.10)

$\therefore \quad q_B = 0.763 \times 2650 \times (9.81 \times 0.3 \times 10^{-3})^{1.5} (1.65)^{1/2}$

$= 0.415$ N/m/s

Suspended Load

At the advanced stage of bed load movement the average shear stress is relatively high and finer particles may move into suspension. With the increase in the shear stress, coarser fractions of the bed material will also move into suspension. The particles in suspension move with a velocity almost equal to the flow velocity. It is also evident that the concentration of sediment particles will be maximum at or near the bed and that it would decrease as the distance from the bed increases. The concentration of suspended sediment is generally expressed as follows:

Volume concentration: The ratio of absolute volume of solids and the volume of sedi- ment-water mixture is termed the volume concentration and can be expressed as percentage by volume. 1 % of volume concentration equals 10,000 ppm by volume.

Weight concentration: The ratio of weight of solids and the weight of sediment-water mixture is termed the weight concentration and is usually expressed in parts per million (ppm).

Variation of Concentration of Suspended Load

Starting from the differential equation for the distribution of suspended material in the vertical and using an appropriate diffusion equation, Rouse (18) obtained the following equation for sediment distribution (i.e., variation of sediment concentration along a vertical):

$$\frac{C}{C_a} = \left[\frac{h-y}{y} \cdot \frac{a}{h-a}\right]^{Z_o} \qquad (7.34)$$

where, C = the sediment concentration at a distance y from the bed,

C_a = the reference concentration at y = a, h = the depth of flow,

$Z_o = \frac{w_o}{U_* k}$ and is the exponent in the sediment distribution equation,

w_o = the fall velocity of the sediment particles, and k = Karman's constant.

Rouse's equation, Eq. (7.34), assumes two-dimensional steady flow, constant fall velocity and fixed Karman's constant. However, it is known that the fall velocity as well as Karman's constant vary with concentration and turbulence. Further, a knowledge of some reference concentration C_a at y = a is required for the use of Eq. (7.34).

Knowledge of the velocity distribution and the concentration variation (Fig. 7.11) would enable one to compute the rate of transport of suspended load q_s. Consider a strip of unit width and thickness dy at an elevation y. The volume of suspended load transported past this strip in
a unit time is equal to $\frac{1}{100}$ Cudy.

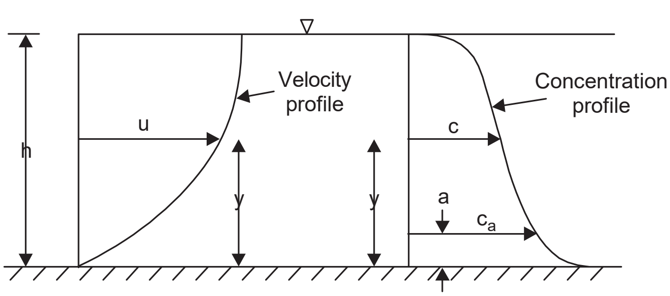


Fig. 7.11 Variation of velocity of flow and sediment concentration in a vertical

Here, C is the volume concentration (expressed as percentage) at an elevation y where the velocity of flow is u. Thus,

$$q_s = \frac{\rho_s g}{100}\int_a^h Cu\,dy \qquad (7.35)$$

where, q_s is the weight of suspended load transported per unit width per unit time. Since the suspended sediment moves only on top of the bed layer, the lower limit of integration, a, can be considered equal to the thickness of the bed layer, i.e., 2d.

Instead of using the curves of the type shown in Fig. 7.11, one may use a suitable velocity distribution law and the sediment distribution equation, Eq. (7.34). For the estimation of the reference concentration C_a appearing in Eq. (7.34), Einstein (17) assumed that the average concentration of bed load in the bed layer equals the concentration of suspended load at y = 2d. This assumption is based on the fact that there will be continuity in the distribution of suspended load and bed load. Making use of suitable velocity distribution laws, the velocity of the bed layer was determined as 11.6 u_*' and as such the concentration in the bed layer was obtained as $\frac{(q_B/\rho_s g)}{(11.6U_*')(2d)}$. Hence, the reference concentration C_a (in per cent) at y = 2d is given as

$$C_a = C_{2d} = \frac{(q_B/\rho_s g)}{(23.2U_*')(d)} \times 100 \qquad (7.36)$$

Equation (7.35) can now be integrated in a suitable manner.

Example 7.8 Prepare a table for the distribution of sediment concentration in the vertical for Example 7.2. Assume fall velocity of the particles as 0.01 m/s.

Solution: From the solution of Example 7.2 and 7.3,

$q_B = 0.286$ N/m/s

and $R' = 0.651$ m

Using Eq. (7.36)

$$C_a = \frac{(q_B/\rho_s g)}{23.2U_*'d} \times 100$$

$$= \frac{0.286 \times 100}{(2650 \times 9.81) \times 23.2 \times (9.81 \times 0.651 \times 1.6 \times 10^{-4})^{1/2}(0.3 \times 10^{-3})}$$

$= 5\%$

Now $\frac{C}{C_a} = \left(\frac{h-y}{y}\,\frac{a}{h-a}\right)^{Z_o}$

$a = 2d = 2 \times 0.3 \times 10^{-3}$ m

$$Z_o = \frac{w_o}{U_* k} = \frac{0.01}{(9.81 \times 2.5 \times 1.6 \times 10^{-4})^{1/2} \times 0.4} = 0.4$$

$$\frac{C}{C_a} = \left(\frac{2.8 - y}{y}\,\frac{2 \times 0.3 \times 10^{-3}}{\ldots}\right)^{0.4}$$

$5.0 = \frac{M}{N}$ y $2.8 (2 \; 0.3 \; 10^{3}) \frac{P}{Q}$

$\frac{2.8 \; y}{}$ 0.4

$C = 0.17$ y $\frac{J}{K}$

The variation of C with y can now be computed as shown in the following table:

y (m)	0.1	0.2	0.5	1.0	1.5	2.0	2.5	2.7	2.8
C (%)	0.635	0.474	0.313	0.215	0.161	0.118	0.0728	0.0455	0
C (ppm)	6350	4740	3130	2150	1610	1180	728	455	0

7.5.3. Total Bed-Material Load

The total bed-material load can be determined by adding together the bed load and the suspended load. There is, however, another category of methods too for the estimation of the total bed-material load. The supporters of these methods argue that the process of suspension is an advanced stage of tractive shear along the bed, and, therefore, the total load should be related to the shear parameter. One such method is proposed by Engelund and Hansen (19) who obtained a relationship for the total bed-material load q_T (expressed as weight per unit width per unit time) by relating the sediment transport to the shear stress and friction factor f. The relationship is expressed as

$$f\phi_T = 0.4\,\tau_*^{5/2} \qquad (7.37)$$

where,

$$\phi_T = \frac{q_T}{\rho_s g^{3/2} d^{3/2}\sqrt{\Delta\rho_s/\rho}} \qquad (7.38)$$

and

$$f = \frac{8\,gRS}{U^2} \qquad (7.39)$$

The median size d_{50} is used for d in the above equation.

Example 7.9 Determine the total bed-material transport rate for Example 7.2.

Solution: Using Eqs. (7.37) and (7.39)

$$f\phi_T = 0.4\,\tau_*^{5/2}$$

$$\frac{8\,gRS}{U^2}\,\phi_T = 0.4\left(\frac{RS}{\Delta_s d}\right)^{5/2}$$

or $$\phi_T = 0.4\left(\frac{2.5 \times 1.6 \times 10^{-4}}{1.65 \times 0.3 \times 10^{-3}}\right)^{5/2} \times \left(\frac{0.95 \times 0.95}{8 \times 9.81 \times 2.5 \times 1.6 \times 10^{-4}}\right)$$

$$\therefore \quad \frac{q_T}{\rho_s g^{3/2} d^{3/2}\sqrt{\Delta\rho_s/\rho}} = 6.75$$

∴ $q_T = 6.75 \times 2650 \times (9.81 \times 0.3 \times 10^{-3})^{3/2} (1.65)^{1/2}$
$= 3.67$ N/m/s

DESIGN OF STABLE CHANNELS

GENERAL

Surface water for irrigation is conveyed from its source to the field by means of canals or channels. These channels generally have alluvial boundaries and carry sediment-laden water. A hydraulic engineer is concerned with the design, construction, operation, maintenance, and improvement of such channels.

Lane (1) gave the definition of stable channel as follows:

''A stable channel is an unlined earth channel : (a) which carries water, (b) the banks and bed of which are not scoured objectionably by moving water, and (c) in which objectionable deposits of sediment do not occur''. This means that over a long period, the bed and banks of a stable channel remain unaltered even if minor deposition and scouring occur in the channel. Obviously, silting and scouring in a stable channel balance each other over a long period of time.

An irrigation channel can have either a rigid boundary or one consisting of alluvial material. These channels may have to carry either clear water or sediment-laden water. Accordingly, there can be four different types of problems related to the design of a stable channel.

Rigid-boundary (i.e., non-erodible) channels carrying clear water,

Rigid-boundary channels carrying sediment-laden water,

Alluvial channels carrying clear water, and

Aluvial channels carrying sediment-laden water.

The design of a stable channel aims at obtaining the values of mean velocity, depth (or hydraulic radius), width and slope of the channel for known values of discharge Q, sediment discharge Q_T, sediment size d, and the channel roughness characteristics without causing undue silting or scouring of the channel bed.

RIGID BOUNDARY CHANNELS CARRYING CLEAR WATER

Channels of this type have rigid, i.e., non-erodible, boundaries like rock cuts or artificial lining. These channels reduce the seepage loss and thus conserve water and reduce the water logging of the lands adjacent to the channel. Cost of operation and maintenance of lined channels are also less. Lined channel sections are relatively more stable.

The design of such channels is based on the Manning's equation combined with the continuity equation. Thus,

$$Q = \frac{1}{n} AR^{2/3} S^{1/2} \quad (8.1)$$

in which, A is the area of cross-section of flow and the hydraulic radius R = A/P. Here, P is the wetted perimeter. Equation (8.1) can also be written as

$$Q = \frac{A^{5/3} S^{1/2}}{nP^{2/3}} \quad (8.2)$$

This means that for specified values of Manning's n and the slope S, the discharge Q is maximum for a given area of cross-section A when the wetted perimeter P is minimum. A channel section with the minimum wetted perimeter for a given cross-sectional area A is said to be the mostefficient hydraulic section or, simply, the best hydraulic section. Since a circle has the least perimeter for a given area, the circular section is the most efficient section. But, constructiondifficulties in having a circular section rule out the possibilities of having a circular channel in most cases. Thus, the problem reduces to determining the geometric elements of the most efficient hydraulic section for a specified geometric shape.

Considering a rectangular section of width B and depth of flow h, one can determine the dimensions of the most efficient rectangular section as follows:

$A = Bh$

$P = B + 2h$

$= \frac{A}{h} + 2h$

For P to be minimum, $dP \quad A + 2 = 0$

dh $\quad h^2$

i.e., $\quad A = 2h^2$

or $\quad B = 2h$

$\therefore \quad P = 4h$

Thus, $\quad R = \frac{h}{2}$

Likewise, such relationships for the geometric elements of other shapes, summarized in Table 8.1, can be determined.

In practice, the sharp corners in a cross-section are rounded so that these may not become zones of stagnation. Sometimes, the side slopes may also have to be adjusted depending upon the type of bank soil. In India, lined channels carrying discharges less than 55 m^3/s are generally of triangular section of the permissible side slope and rounded bottom (Fig. 8.1). A trapezoidal section with rounded corners (Fig. 8.2) is adopted for lined channels carrying discharges larger than 55 m^3/s. The side slopes depend on the properties of the material through which the channel is to pass. Table 8.2 gives the recommended values of the side slopes for channels excavated through different types of material.

To avoid damage to the lining, the maximum velocity in lined channels is restricted to 2.0 m/s. Thus, the design is based on the concept of a limiting velocity. The expressions for geometric elements of the lined channel section, Figs. 8.1 and 8.2, can be written as follows:

Table. 8.1 Geometric elements of the most efficient hydraulic sections (2)

Shape of cross-section	Area A	Wetted perimeter P	Hydraulic radius R	Water surface width T	Hydraulic depth D
Rectangle	$2h^2$	4h	0.500h	2h	h
Triangle (side slope 1 : 1)	h^2	2.83h	0.354h	2h	0.500h
Trapezoid (half of hexagon)	$1.73h^2$	3.46h	0.500h	2.31h	0.750h
Semicircle	$0.5\pi h^2$	πh	0.500h	2h	$0.250\pi h$
Parabola ($T = 2\sqrt{2}h$)	$1.89h^2$	3.77h	0.500h	2.83h	0.667h

Table 8.2 Suitable side slopes for channels excavated through different types of material (3)

Material	Side slopes (H : V)
Rock	Nearly vertical
Muck and peat soil	0.25 : 1
Stiff clay or earth with concrete lining	0.5 : 1 to 1 : 1
Earth with stone lining	1 : 1
Firm clay	1.5 : 1
Loose, sandy soil	2 : 1

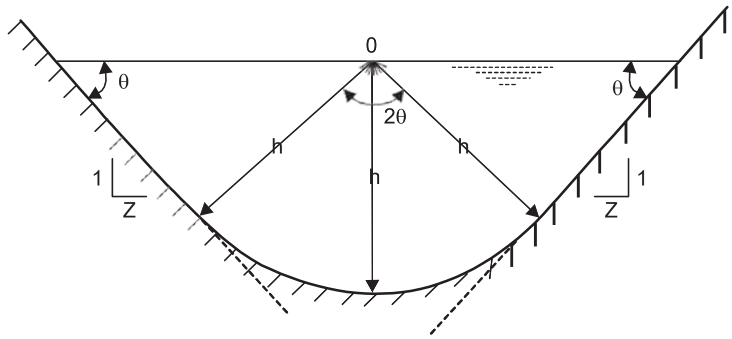


Fig. 8.1 Lined channel section for $Q < 55\ m^3/s$

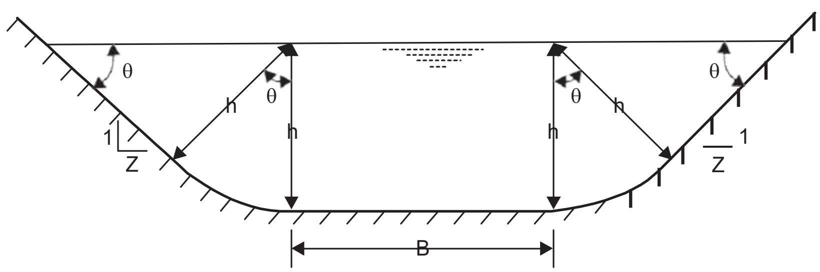


Fig. 8.2 Lined channel section for $Q > 55\ m^3s$

For triangular section, Fig. 8.1,

Area, $A = 2\left(\frac{1}{2}h^2 \cot\theta\right) + \frac{1}{2}h^2(2\theta)$

$= h^2(\theta + \cot\theta)$

Wetted perimeter, $P = 2h\cot\theta + h(2\theta)$

$= 2h(\theta + \cot\theta)$

Hydraulic radius $R = \dfrac{A}{P} = \dfrac{h^2(\theta + \cot\theta)}{2h(\theta + \cot\theta)} = \dfrac{h}{2}$

Similarly, for trapezoidal section, Fig. 8.2,

$A = Bh + 2\left(\frac{1}{2}h^2 \cot\theta\right) + 2\left(\frac{1}{2}h^2\theta\right)$

$= Bh + h^2(\theta + \cot\theta)$

$P = B + 2h(\theta + \cot\theta)$

$R = \dfrac{Bh + h^2(\theta + \cot\theta)}{B + 2h(\theta + \cot\theta)}$

In all these expressions for A, P, and R, the value of θ is in radians. For designing alined channel, one needs to solve these equations alongwith the Manning's equation. For given Q, n, S, and A, and R expressed in terms of h for known Q, the Manning's equation will yield,for triangular section, an explicit relation for h as shown below :

$$Q = \frac{1}{n}[h^2(\theta + \cot\theta)]\left(\frac{h}{2}\right)^{2/3} S^{1/2}$$

$$\therefore \qquad h = \left[\frac{nQ(2)^{2/3}}{\sqrt{S}\,(\theta + \cot\theta)} \right]^{3/8} \tag{8.3}$$

However, in case of trapezoidal section, Fig. 8.2, the design calculations would start with an assumed value of velocity (less than the maximum permissible velocity of 2.0 m/s) and the expression for h will be in the form of a quadratic expression as can be seen from the following :

From the Manning's equation

$$R = \left(\frac{Un}{\sqrt{S}} \right)^{3/2}$$

$$\therefore \qquad \left(\frac{Un}{\sqrt{S}} \right)^{3/2} [B + 2h\,(\theta + \cot\theta)] = A = \frac{Q}{U}$$

$$\therefore \qquad B = \frac{Q}{U}\left(\frac{\sqrt{S}}{Un} \right)^{3/2} - 2h\,(\theta + \cot\theta)$$

On substituting this value of B in the expression for area of flow A, one gets,

$$h\,\frac{Q}{U}\left(\frac{\sqrt{S}}{Un} \right)^{3/2} - 2h^2\,(\theta + \cot\theta) + h^2\,(\theta + \cot\theta) = A = \frac{Q}{U}$$

$$h^2(\theta + \cot\theta) - h\,\frac{Q}{V}\left(\frac{\sqrt{S}}{Vn}\right)^{3/2} + \frac{Q}{V} = 0$$

$$\therefore \quad h = \frac{\frac{Q}{V}\left(\frac{\sqrt{S}}{Vn}\right)^{3/2} \pm \sqrt{\frac{Q^2}{V^2}\left(\frac{\sqrt{S}}{Vn}\right)^{3} - 4(\theta + \cot\theta)\frac{Q}{V}}}{2(\theta + \cot\theta)}$$

Therefore, in order to have a feasible solution,

$$\frac{Q^2}{V^2}\left(\frac{\sqrt{S}}{Vn}\right)^{3} \geq 4(\theta + \cot\theta)\frac{Q}{V}$$

i.e., $$\frac{S^{3/2}}{V^4} \geq \frac{4n^3(\theta + \cot\theta)}{Q}$$

or $$V \leq \left[\frac{QS^{3/2}}{4n^3(\theta + \cot\theta)}\right]^{1/4} \qquad (8.4)$$

This means that for designing a trapezoidal section for a lined channel, the velocity will have to be suitably chosen so as not to violate the above criterion in order to have a feasible solution (see Example 8.2 for illustration).

Example 8.1 A lined canal (n = 0.015) laid at a slope of 1 in 1600 is required to carry a discharge of 25 m^3/s. The side slopes of the canal are to be kept at 1.25 H : 1 V. Determine thedepth of flow.

Solution: Since Q < 55 m^3/s, a triangular section with rounded bottom, Fig. 8.1, is considered suitable. Here,

$\cot\theta = 1.25$

$\therefore \quad \theta = 38.66°$ or 0.657 radian Thus, from Fig. 8.1, $A = h^2(\theta + \cot\theta) = h^2(0.675 + 1.25)$

$= 1.925h^2$

and $\quad P = 2h(\theta + \cot\theta) = 2h(0.675 + 1.25)$

$= 3.85h$

$$\therefore \quad R = \frac{1.925h^2}{3.85h} = \frac{h}{2}$$

From the Manning's equation,

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

$$\text{Hence, } 25 = \frac{1.925h^2 \left(\frac{h}{2}\right)^{2/3} \left(\frac{1}{1600}\right)^{1/2}}{0.015}$$

$$\therefore \quad h^{8/3} = \frac{25 \times 0.015 \times 2^{2/3} \times \left(\frac{1600}{2}\right)}{40 \times 1.925}$$

$\therefore \quad h = 2.57$ m.

Example 8.2 Design a lined channel to carry a discharge of 300 m^3/s through an alluvium whose angle of repose is 31°. The bed slope of the channel is 7.75×10^{-5} and Manning's n for the lining material is 0.016.

Solution: Since $Q > 55$ m^3/s, trapezoidal section with rounded corners, Fig. 8.2, is to be designed. Here,

Side slope $\theta = 31° = 0.541$ radians

$\therefore \quad \cot \theta = 1.664$

$\therefore \quad \theta + \cot \theta = 2.205$

$\therefore \quad A = Bh + 2.205\, h^2$

$P = B + 4.41h$

Adopting $U = 2$ m/s

$$A = \frac{300}{2} = 150 \text{ m}^2$$

$\therefore \quad Bh + 2.205h^2 = 150$

and
$$R = \left(\frac{Un}{\sqrt{S}}\right)^{3/2} = \left(\frac{2 \times 0.016}{\sqrt{7.75 \times 10^{-5}}}\right)^{3/2} = 6.93 \text{ m}$$

$\therefore \quad 6.93\,(B + 4.41h) = 150$

or $\quad B = 21.645 - 4.41h$

$\therefore \quad 21.645h - 4.41h^2 + 2.205h^2 = 150$ or $\quad 2.205h^2 - 21.645h + 150 = 0$

$$\therefore \quad h = \frac{21.645 \pm \sqrt{(21.645)^2 - 4 \times 2.205 \times 150}}{4.41}$$

Obviously, the roots of h are imaginary. Using the criterion, Eq. (8.4), one gets,

$$U \le \left[\frac{QS^{3/2}}{4n^3(\theta + \cot \theta)}\right]^{1/4}$$

$$\le \left[\frac{300 \times (7.75 \times 10^{-5})^{3/2}}{4(0.016)^3(2.205)}\right]^{1/4}$$

≤ 1.543 m/s

$\therefore$ Adopt $U = 1.5$ m/s

$\therefore \quad A = 200 \text{ m}^2$

$$R = \left(\frac{1.5 \times 0.016}{\sqrt{7.75 \times 10^{-5}}}\right)^{3/2} = 4.50 \text{ m}$$

$4.50(B + 4.41h) = 200$

$\therefore \quad B = 44.44 - 4.41h$

Again, using $Bh + 2.205h^2 = A$, one gets, $44.44h - 4.41h^2 + 2.205h^2 = 200$

or $2.205h^2 - 44.44h + 200 = 0$

$$\therefore \quad h = \frac{44.44 \pm \sqrt{(44.44)^2 - 4 \times 2.205 \times 200}}{4.41}$$

= 13.37 m or 6.784 m

$\therefore \quad B = 44.44 - 4.41h$

= 14.52 m for h = 6.784 m

Other value of h (= 13.37 m) gives negative value of B which is meaningless.

$\therefore \quad B = 14.52$ m and $h = 6.784$ m.

RIGID BOUNDARY CHANNELS CARRYING SEDIMENT–LADEN WATER

These channels are to be designed in such a way that the sediment in suspension does not settle on the channel boundary. The design is, therefore, based on the concept of minimum permissible velocity which will prevent both sedimentation as well as growth of vegetation. In general, velocities of 0.7 to 1.0 m/s will be adequate for this purpose if the sediment load is small. If the sediment concentration is large, Fig. 8.3 can be used to ensure that the sediment does not deposit. In Fig. 8.3, C_s is the concentration of sediment in ppm (by volume), f_b the friction factor of the channel bed, D_o the central depth, T the top width and S_c equals $S/(\Delta\gamma_s/\gamma)$.

If the designed channel section is not able to carry the specified sediment load, the slope S ofthe channel is increased.

Example 8.3 A rectangular channel 5 m wide is to carry 2.5 m^3/s on a slope of 1 in 2000 at a depth of 0.75 m. It is expected that fine silt of 0.04 mm size will enter the channel. What is the maximum concentration of this sediment that can be allowed into the channel without causing objectionable deposition ? Assume that the fall velocity of the given sediment in water is 1.5 mm/s, kinematic viscosity of water is 10^{-6} m^2/s, and specific gravity of the sediment is 2.65.

Solution: Since the wall and the bed of the channel are of the same material, the friction factor for the channel bed f_b is given as

$$f_b = \frac{8gRS}{U^2}$$

Since $U = \frac{2.5}{5 \times 0.75} = 0.7143$ m/s

and $R = \frac{5 \times 0.75}{5 + 2 \times 0.75} = 0.5769$ m

$$\therefore \quad f_b = \frac{8 \times 9.81 \times 0.5769 \times (1/2000)}{(0.7143)^2} = 0.0444$$

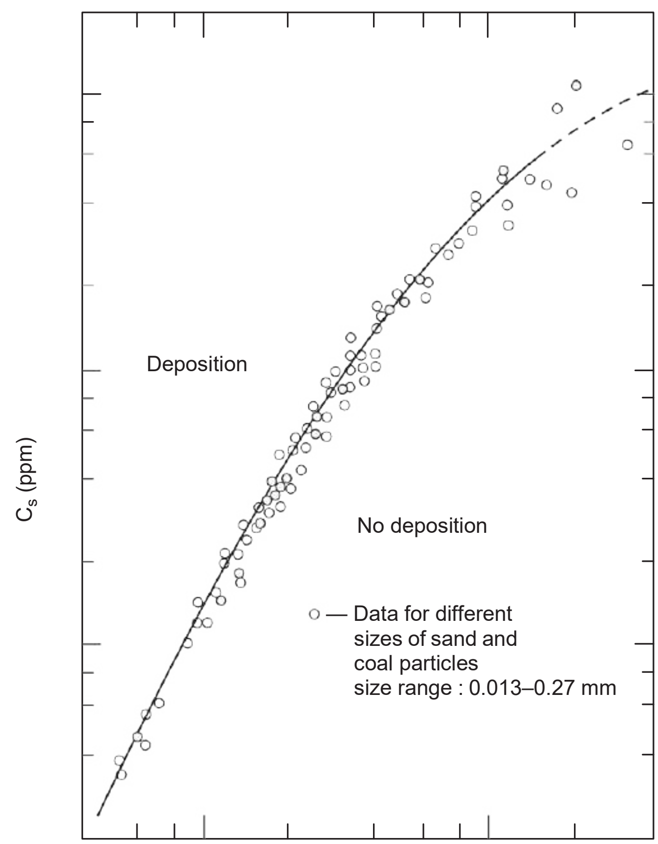
$$S_c = S/(\Delta\gamma_s/\gamma) = \frac{1}{2000 \times 1.65} \approx \frac{1}{3300}$$

Now $\frac{qS_c^{2.5}}{2}$ 1 / 0.6 A 2

fb (w0 d/) HTD0 K
(2.5 / 5.0)(1 / 3300)^2.5 5 0.75 2

= (10^6)(0.0444)^2 (1.5 10^3 0.4 10^3 / 10^6)^0.6 5 0.75 = 2.2

Therefore, from Fig. 8.3, maximum concentration for no deposition, C_S = 500 ppm



4
10

3
10

2
10

1
2×10
0.4 1.0 10 40
2.5
q Sc ×
f
2 1 (w d/
0.6

×(A)^2 TD0

b 0)

Fig. 8.3 Relation for the limiting concentration of suspended sediment in channels (4)

ALLUVIAL CHANNELS CARRYING CLEAR WATER

Compared to the design methods used for rigid boundary channels, the design of stable alluvial channels is more complex. Alluvial channels carrying clear water should be designed such that the erodible material of the channel boundary is not scoured. The design of such channels is, therefore, based on the concept of tractive force.

Scour on a channel bed occurs when the tractive force on the bed exerted by the flow is adequate to cause the movement of the bed particles. A sediment particle resting on the sloping side of a channel will move due to the resultant of the tractive force in the flow direction and the component of gravitational force which makes the particle roll or slide down the side slope. If the tractive force acting on the bed or the resultant of the tractive force and the component of the gravitational force both acting on the side slopes is larger than the force resisting the movement of the particle, erosion starts. The following method of design of stable alluvial channels based on this principle was proposed by Lane (1) and is also known as the USBR method.

In uniform flow, the average tractive stress, τ_0 is given as

$$\tau_o = \rho gRS \qquad (8.5)$$

The shear stress is not uniformly distributed over the channel perimeter. Figure 8.4 shows the variation of the maximum shear stresses acting on the side, τ_{sm} and the bed, τ_{bm} of a channel. It should be noted that for the trapezoidal channels, the maximum tractive shear on the sides is approximately 0.76 times the tractive shear on the channel bed.

For a particle resisting on a level or mildly sloping bed, on can write the following expression for the incipient motion condition :

$$\tau_{bl}\, a = W_s \tan \theta \qquad (8.6)$$

where, W_s is the submerged weight of the particle and a, the effective area of the particle over which the tractive stress τ_{bl} is acting, and θ is the angle of repose for the particle. τ_{bl} is, obviously, the critical shear stress τ_c for the bed particles. Since the aim is to avoid the movement of the movement of the particles (5), τ_{bl} may be kept less than τ_c , say 0.9 τ_c.

For a particle resting on the sloping side of a channel (Fig. 8.5), the condition for the incipient motion is

$$(W_s \sin \alpha)^2 + (\tau_{sl}\, a)^2 = (W_s \cos \alpha \tan \theta)^2$$

$$\Rightarrow \qquad W_s^2 \sin^2 \alpha + (\tau_{sl}\, a)^2 = W_s^2 \cos^2 \alpha \tan^2 \theta$$

$$\text{or} \qquad \tau_{sl} = \frac{W_s}{a} \cos \alpha \tan \theta \sqrt{1 - \frac{\tan^2 \alpha}{\tan^2 \theta}} \qquad (8.7)$$

On combining Eqs. (8.6) and (8.7)

$$k = \frac{\tau_{sl}}{\tau_{bl}} = \cos \alpha \sqrt{1 - \frac{\tan^2 \alpha}{\tan^2 \theta}} \qquad (8.8)$$

i.e., $\tau_{sl} = k\, \tau_{bl} = 0.9k\, \tau_c$

For non-scouring condition, the design rules become

$$\tau_{bm} \le \tau_{bl}$$

$$\tau_{sm} \le \tau_{sl}$$

Lane also observed that the channels, which are curved in alignment, scour more readily. He, therefore, suggested some correction factors which should be multiplied with the critical value of tractive stress (1). The values of critical shear stress (and also τ_{bl}) for bed and sides of curved channels are given in Table 8.3.

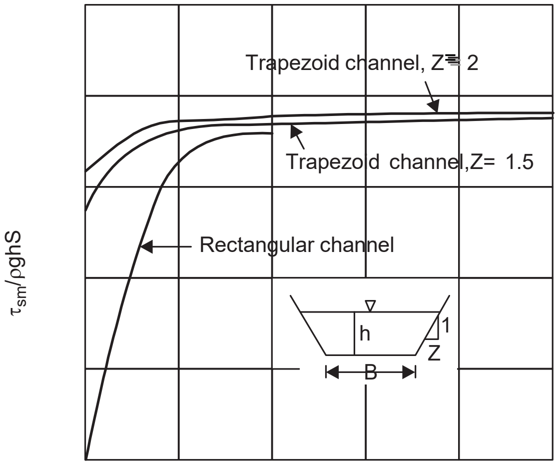
1.0

0.8

0.6

0.4

0.2



0
0 2 4 6 8 10
B/h
(a) Channel sides

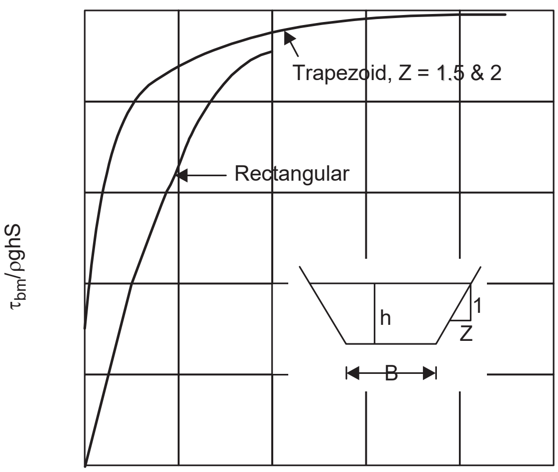
1.0

0.8

0.6

0.4

0.2



0
0 2 4 6 8 10
B/h
(b) Channel bed

Fig. 8.4 Maximum shear stress on (a) sides and (b) bed of smooth channels in uniform flow

Table 8.3 Critical shear stress and the values of τ_{bl} for curved channels

Type of channel	Critical shear stress (τ_c)	τ_{bl}

Straight channels	$\square_c$	0.900 $\square_c$
Slightly curved channels	0.90 $\square_c$	0.810 $\square_c$
Moderately curved channels	0.75 $\square_c$	0.675 $\square_c$
Very curved channels	0.60 $\square_c$	0.540 $\square_c$

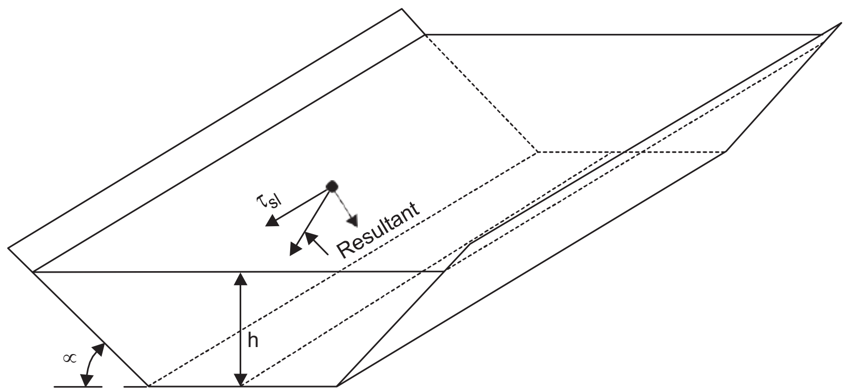


Fig. 8.5 Forces causing movement of a particle resting on a channel bank

Example 8.4 Design a trapezoidal channel (side slopes 2H : 1V) to carry 25 m^3/s of clear water with a slope equal to 10^{-4}. The channel bed and banks comprise gravel (angle of repose = 31°) of size 3.0 mm. The kinematic viscosity of water may be taken as 10^{-6} m^2/s.

Solution: From Eq. (7.2),

$$R_0^* = \sqrt{\frac{\Delta\rho_s g d^3}{\rho\nu^2}} = \sqrt{\frac{(1.65)(9.81)(3.0\times10^{-3})^3}{(10^{-6})^2}}$$

$\therefore \quad R_0^* = 661.1$

From Fig. 7.3, $\tau_c^* = 0.045$

$\therefore \quad \tau_c = (0.045)\,\Delta\rho_s\, gd = 0.045 \times 1650 \times 9.81 \times (3.0 \times 10^{-3})$

$= 2.185$ N/m^2

Taking $\tau_{bl} = 0.9\,\tau_c$

$= 1.97$ N/m^2

$\therefore \quad \tau_{bm} = 1.97$ N/m^2

Now, $$k = \cos\alpha\sqrt{1-\frac{\tan^2\alpha}{\tan^2\theta}} = \frac{2}{\sqrt{5}}\sqrt{1-\frac{(0.5)^2}{(0.6)^2}} = 0.494$$

$\therefore \quad \tau_{sl} = 0.494 \times \tau_{bl} = 0.494 \times 1.97 = 0.973$ N/m^2 $= \tau_{sm}$

Rest of the computations are by trial and can be carried out as follows : Assume B/h = 10

Therefore, from Fig. 8.4,

$$\frac{\tau_{sm}}{\rho g h S} = 0.78$$

$$h = \frac{0.973}{9810 \times 10^{-4} \times 0.78} = 1.27 \text{ m}$$

Also from Fig. 8.4,

$$\frac{\tau_{bm}}{\rho g h S} = 0.99$$

$$\therefore \quad h = \frac{1.97}{9810 \times 10^{-4} \times 0.99} = 2.03 \text{ m}$$

Choosing the lesser of the two values of h

h = 1.27 m

$\therefore \quad B = 10h = 12.7$ m

$A = Bh + 2h^2 = 12.7 \times 1.27 + 2 \times (1.27)^2 = 19.355 \text{ m}^2$

$P = B + 2\sqrt{5}\, h = 12.7 + (2 \times \sqrt{5} \times 1.27) = 18.38$ m

R = 1.053 m

$$\text{and} \quad n = \frac{d^{1/6}}{25.6} = \frac{(3 \times 10^{-3})^{1/6}}{25.6} = 0.0148$$

$$\therefore \quad Q = \frac{1}{n} AR^{2/3} S^{1/2} = \frac{1}{0.0148} \times 19.355 \times (1.053)^{2/3} (10^{-4})^{1/2}$$

$= 13.5 \text{ m}^3/\text{s}$

Since this value of Q is less than the given value, another value of B/h, say, 20.0 is assumed. Using Fig. 8.4, it will be seen that h = 1.27 m.

$\therefore \quad B = 25.4$ m

$\therefore \quad A = 35.484 \text{ m}^2$, P = 31.08 m, R = 1.142 m and $Q = 26.195 \text{ m}^3/\text{s}$

This value of Q is only slightly greater than the desired value 25.00 m^3/s.

Hence, B = 25.4 m and h = 1.27 m. The trial calculations can be done in a tabular form as shown below:

B/h	$\frac{\tau_{sm}}{\gamma hS}$	$\frac{\tau_{bm}}{\gamma hS}$	h(m)	B(m)	$A(m^2)$	P(m)	R(m)	$Q(m^3/s)$
10.0	0.78	0.99	1.27	12.7	19.355	18.38	1.053	13.500
20.0	0.78	0.99	1.27	25.4	35.484	32.08	1.142	26.195

ALLUVIAL CHANNELS CARRYING SEDIMENT-LADEN WATER

The cross-section of a stable alluvial channel would depend on the flow rate, sediment transport rate, and the sediment size. There are two methods commonly used for the design of alluvial channels carrying sediment-laden water. The first is based on the 'regime' approach in which a set of empirical equations is used. These equations have been obtained by analysing the data of stable field channels. A more logical method of design of stable alluvial channel should include the sediment load also.

Regime Methods

Regime Methods for the design of stable channels were first developed by the British engineers working for canal irrigation in India in the nineteenth century. At that time, the problem of sediment deposition was one of the major problems of channel design in India. In order to finda solution to this problem, some of the British engineers studied the behaviour of such stretches of the existing canals where the bed was in a state of stable equilibrium. The stable reaches had not required any sediment clearance for several years of the canal operation. Such channels were called regime channels. These channels generally carried a sediment load smaller than 500 ppm. Suitable relationships for the velocity of flow in regime channels were evolved. These relationships are now known as regime equations which find acceptance in other parts of the world as well. The regime relations do not account for the sediment load and, hence, should be considered valid when the sediment load is not large.

Kennedy's Method

Kennedy (6) collected data from 22 channels of Upper Bari Doab canal system in Punjab. His observations on this canal system led him to conclude that the sediment in a channel is kept in suspension solely by the vertical component of the eddies which are generated on the channel bed. In his opinion, the eddies generating on the sides of the channel had horizontal movement for greater part and, therefore, did not have sediment supporting power. This means that the sediment supporting power of a channel is proportional to its width (and not wetted perimeter).

On plotting the observed data, Kennedy obtained the following relation, known as Kennedy's equation.

$U_o = 0.55h^{0.64}$ (8.9)

Kennedy termed U_o as the critical velocity[1] (in m/s) defined as the mean velocity which will not allow scouring or silting in a channel having depth of flow equal to h (in metres). This equation

is, obviously, applicable to such channels which have the same type of sediment as was present in the Upper Bari Doab canal system. On recognising the effect of the sediment size on the critical velocity, Kennedy modified Eq. (8.9) to

$U = 0.55mh^{0.64}$ (8.10)

in which m is the critical velocity ratio and is equal to U/U_o. Here, the velocity U is the critical velocity for the relevant size of sediment while U_o is the critical velocity for the Upper Bari Doab sediment. This means that the value of m is unity for sediment of the size of Upper Bari Doab sediment. For sediment coarser than Upper Bari Doab sediment, m is greater than 1 while for sediment finer than Upper Bari Doab sediment, m is less than 1. Kennedy did not try to establish any other relationship for the slope of regime channels in terms of either the critical velocity or the depth of flow. He suggested the use of the Kutter's equation along with the Manning's roughness coefficient. The final results do not differ much if one uses the Manning's equation instead of the Kutter's equation. Thus, the equations

$U = 0.55mh^{0.64}$ (8.10)

$Q = AU$ (8.11)

and $U = \frac{1}{n} R^{2/3} S^{1/2}$ (8.12)

enable one to determine the unknowns, B, h, and U for given Q, n, and m if the longitudinal slope S is specified.

The longitudinal slope S is decided mainly on the basis of the ground considerations. Such considerations limit the range of slope. However, within this range of slope, one can obtain different combinations of B and h satisfying Eqs. (8.10) to (8.12). The resulting channel section can vary from very narrow to very wide. While all these channel sections would be able to carry the given discharge, not all of them would behave satisfactorily. Table

8.4 gives valuesof recommended width-depth ratio, i.e., B/h for stable channel (5).

[1]**This critical velocity should be distinguished from the critical velocity of flow in open channels corresponding to Froude number equal to unity.**

Table 8.4 Recommended values of B/h for stable channels

Q (m^3/s)	5.0	10.0	15.0	50.0	100.0	200.0	300.0
B/h	4.5	5.0	6.0	9.0	12.0	15.0	18.0

Several investigations carried out on similar lines indicated that the constant C⍰ and the exponent x in the Kennedy's equation, U = C⍰ mh^x are different for different canal systems. Table 8.5 gives the value of C⍰ and x in the Kennedy's equation for some regions.

Table 8.5 Values of C⍰ and x in the Kennedy's equation fordifferent regions (7)

Region	C⍰	x
Egypt	0.25 to 0.31	0.64 to 0.73
Thailand	0.34	0.66
Rio Negro (Argentina)	0.66	0.44
Krishna River (India)	0.61	0.52
Chenab River (India)	0.62	0.57
Pennar River (India)	0.60	0.64
Shwebo (Burma)	0.60	0.57
Imperial Valley (USA)	0.64 to 1.20	0.61 to 0.64

The design procedure based on Kennedy's theory involves trial. For known Q, n, m, andS, assume a trial value of h and obtain the critical velocity U from the Kennedy's equation [Eq. (8.10)]. From the continuity equation [Eq. (8.11)] one can calculate the area of cross-section Aand, thus, know the value of B for the assumed value of h. Using these values of B and h, compute the mean velocity from the Manning's equation [Eq. (8.12)]. If this value of the meanvelocity matches with the value of the critical velocity obtained earlier, the assumed value of h and the computed value of B provide channel dimensions. If the two velocities do not match,assume another value of h and repeat the calculations.

Ranga Raju and Misri (8) suggested a simplified procedure which does not involve trial. The method is based on the final side slope of 1H : 2V attained by an alluvial channel. During construction, the side slopes of a channel are kept flatter than the angle of repose of the soil. But, after some time of canal running, the side slopes become steeper due to the deposition of sediment. The final shape of the channel cross-section is approximately trapezoidal with side slopes 1H : 2V. For this final cross-section of channel, one can write,

$A = Bh + 0.5h^2 = h^2(p + 0.5)$ (8.13)

$P = B + 2.236h = h(p + 2.236)$ (8.14)

where, p = B/h

Now $R = \frac{h(p + 0.5)}{p + 2.236}$ (8.15)

and $U = \frac{Q}{h^2(p + 0.5)}$ (8.16)

Substituting the value of U and R in the Manning's equation [Eq. (8.10)], one obtains

$$S = \frac{Q^2 n^2 (p + 2.236)^{4/3}}{h^{16/3} (p + 0.5)^{10/3}} \qquad (8.17)$$

Similarly, substituting the value of U in the Kennedy's equation [Eq. (8.10)], one gets,

$$h = \left[\frac{1.818Q}{(p + 0.5)m}\right]^{0.378} \qquad (8.18)$$

On substituting the value of h from Eq. (8.18) in Eq. (8.17), one finally obtains

$$\frac{SQ^{0.02}}{n^2 m^2} = 0.299 \frac{(B/h + 2.236)^{1.333}}{(B/h + 0.5)^{1.313}} \qquad (8.19)$$

Figure 8.6 shows the graphical form of Eq. (8.19).

For given Q, n, m, and a suitably selected value of S, compute $SQ^{0.02}/n^2 m^2$ and read the value of B/h from the Fig. 8.6. From Table 8.4, check if this value of B/h is satisfactory for the given discharge. If the value of B/h needs modification, choose another slope. Having obtained B/h, calculate h from Eq. (8.18) and then calculate B.

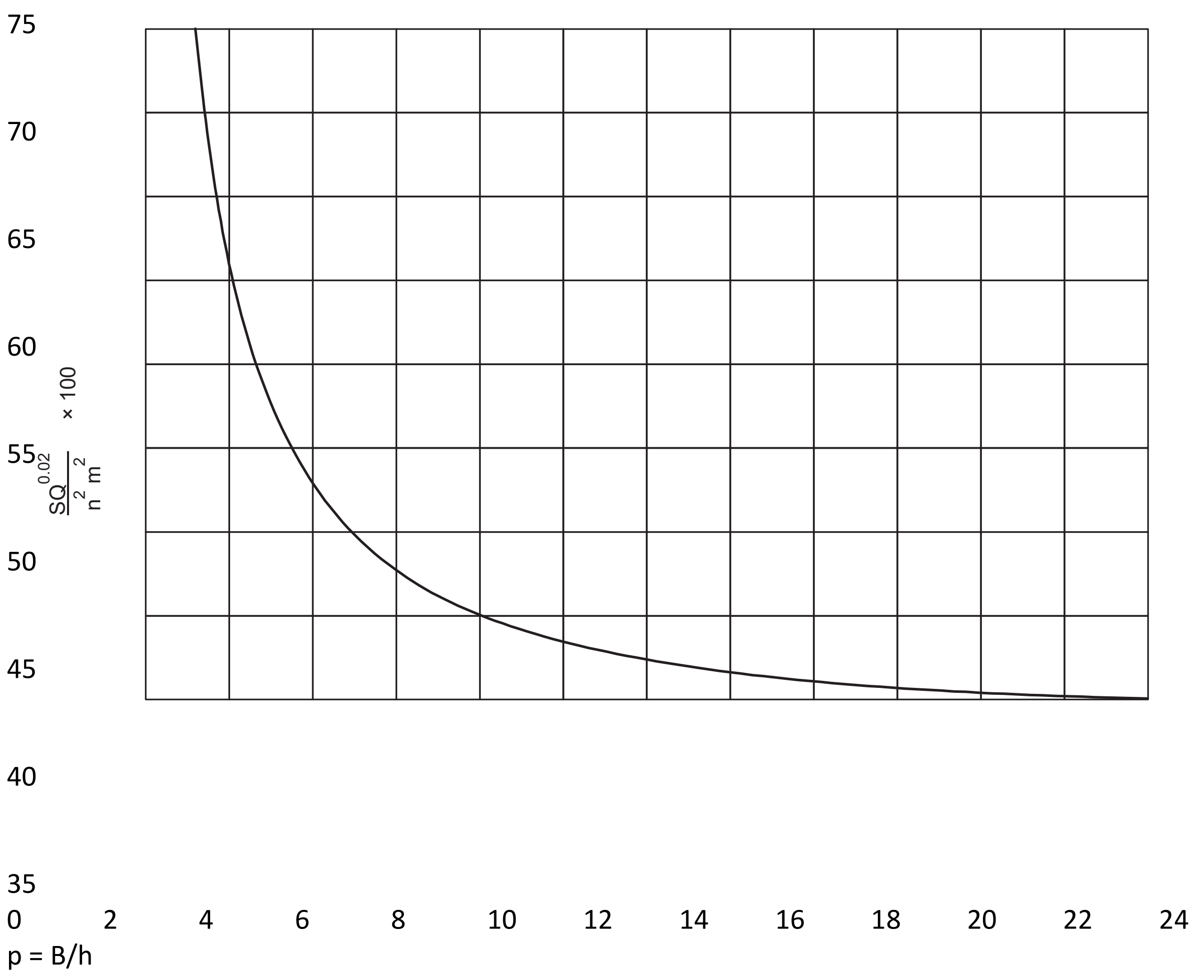


Fig. 8.6 Diagram for design of alluvial channels using the Kennedy's equation (8)

Example 8.5 Design a channel carrying a discharge of 30 m^3/s with critical velocity ratio and Manning's n equal to 1.0 and 0.0225, respectively. Assume that the bed slope is equal to 1 in 5000.

Solution:

Kennedy's method:

Assume h = 2.0 m. From Kennedy's equation [Eq. (8.10)]

$$U = 0.55\, m h^{0.64} = 0.55 \times 1 \times (2.0)^{0.64}$$
$$= 0.857 \text{ m/s}$$

$$\therefore \quad A = Q/U = \frac{30}{0.857} = 35.01 \text{ m}^2$$

For a trapezoidal channel with side slope 1 H : 2V,

$$Bh + \frac{h^2}{2} = B(2.0) + \frac{2 \times 2}{2} = 2B + 2 = 35.01$$

$$\therefore \quad B = 16.51 \text{ m}$$

$$\therefore \quad R = \frac{35.01}{16.51 + 2.0\sqrt{5}} = 16.7 \text{ m}$$

Therefore, from the Manning's equation [Eq. (8.12)],

$$U = \frac{1}{n} R^{2/3} S^{1/2} = \frac{1}{0.0225} (1.67)^{2/3} \left(\frac{1}{5000}\right)^{1/2} = 0.885 \text{ m/s}$$

Since the velocities obtained from the Kennedy's equation and Manning's equation are appreciably different, assume h = 2.25 m and repeat the above steps.

$$U = 0.55 \times 1 \times (2.25)^{0.64} = 0.924 \text{ m/s}$$

$$A = \frac{30}{0.924} = 32.47 \text{ m}^2$$

$$\therefore \quad B(2.25) + (0.5)(2.25)^2 = 32.47$$

$$\therefore \quad B = 13.31 \text{ m}$$

$$R = \frac{32.47}{13.31 \times (\sqrt{5} \times 2.25)} = 1.77 \text{ m}$$

$$U = \frac{1}{0.0225} (1.77)^{2/3} \left(\frac{1}{5000}\right)^{1/2}$$
$$= 0.92 \text{ m/s}$$

Since the two values of the velocities are matching, the depth of flow can be taken as equal to 2.25 m and the width of trapezoidal channel = 13.31 m.

Ranga Raju and Misri's method :

$$\frac{SQ^{0.02}}{n^2 m^2} = \frac{\frac{1}{5000}(30)^{0.02}}{(0.0225)^2 \times (1)^2} = 0.423$$

Hence, from Fig. 8.6,

$$p = \frac{B}{h} = 6.0$$

and
$$h = \left[\frac{1.818\, Q}{(p + 0.5)m}\right]^{0.378} = \left[\frac{1.818 \times 30}{(6.0 + 0.5) \times 1}\right]^{0.378} = 2.235 \text{ m}$$

? B = 6.0 h = 13.41 m

Q 30

and U = ?

h^2 (p ? 0.5) (2.235)^2 (6.0 ? 0.5)

= 0.924 m/s.

Lindley's Method

Lindley (9) was the first to recognise that width, depth, and the slope of a channel can all adjust in an alluvial channel for a given set of conditions. He stated that when an artificial channel is used to carry sediment-laden water, both the bed and banks either scour or silt and thus change depth, gradient, and width until a state of balance is attained at which condition the channel is said to be in regime.

The observed width, slope, and depth of the Lower Chenab canal system were analysed by Lindley (9) using n = 0.025 and side slopes as 0.5 H : 1V. He obtained the following equations :

$U = 0.57\ h^{0.57}$ (8.20)

$U = 0.27\ B^{0.355}$ (8.21)

From Eqs. (8.20) and (8.21), one can get

$B = 7.8\ h^{1.61}$ (8.22)

It should be noted that these equations do not include the effect of sediment size on the multiplying coefficient. Woods (10) also proposed equations similar to Eqs. (8.20) and (8.21).

Lacey's Method

Lacey (11) stated that the dimensions width, depth, and slope of a regime channel to carry a given water discharge loaded with a given sediment discharge are all fixed by nature. Accordingto him, the fundamental requirements for a channel to be in regime are as follows :

The channel flows uniformly in incoherent alluvium. Incoherent alluvium is the loose granular material which can scour or deposit with the same ease. The material may range from very fine sand to gravel, pebbles, and boulders of small size.

The characteristics and the discharge of the sediment are constant.

The water discharge in the channel is constant.

The perfect 'regime' conditions rarely exist. The channels which have lateral restraint (because of rigid banks) or imposed slope are not considered as regime channels. For example, an artificial channel, excavated with width and longitudinal slope smaller than those required, will tend to widen its width and steepen its slope if the banks and bed are of incoherent alluvium and non-rigid. In case of rigid banks, the width is not widened but the slope becomes steeper. Lacey termed this regime as the initial regime. A channel in initial regime is narrower than it would have been if the banks were not rigid. This channel has attained working stability. If the continued flow of water overcomes the resistance to bank erosion so that the channel now has freedom to adjust its perimeter, slope, and depth in accordance with the discharge, the channel is likely to attain what Lacey termed the final regime.

The river bed material may not be active at low stages of the river particularly if the bed is composed of coarse sand and boulders. However, at higher stages, the bed material becomes active, i.e., it starts moving. As such, it is only during the high stages that the river may achieve regime conditions. This fact is utilised in solving problems related to floods in river channels.

Lacey also suggested that for a regime channel the roughness coefficient as well as the critical velocity ratio should be dependent on sediment size alone. However, it is now well known that in a movable bed channel, the total roughness includes grain as well as form roughness. Likewise, the non-silting and non-scouring velocity (included in the critical velocity ratio) shall depend on the sediment load and the size of the sediment.

Lacey felt that the sediment in an alluvial channel is kept in suspension by the vertical components of eddies generated at all points along the wetted perimeter. He, therefore, plotted the available data of regime channels to obtain a relationship between the regime velocity U (in m/s) and the hydraulic radius R (in metres). He, thus, found that $U \propto R^{1/2}$ and that the exponential power did not change with data. He, therefore, formulated

$$U = C'\sqrt{R} \qquad (8.23)$$

in which C' is a proportionality constant.

Including a factor f_1 to account for the size and density of sediment, Lacey finally obtained

$$U = \sqrt{\frac{2}{5} f_1 R} = 0.632\sqrt{f_1 R} \qquad (8.24)$$

The factor f_1 has been named as the silt factor. For natural sediment of relative density equal to 2.65, the silt factor f_1 can be obtained by Lacey's relation

$$f_1 = 1.76\sqrt{d} \qquad (8.25)$$

where, d is the median size of sediment in millimetre.

On plotting Lindley's data and other data of regime channels, Lacey obtained

$$R^{1/2} S = C''$$

and $\quad C' = 10.8 C''^{1/3}$

$$\therefore \quad \frac{U}{R^{1/2}} = 10.8\,(R^{1/2} S)^{1/3}$$

which gives $\quad U = 10.8 R^{2/3} S^{1/3} \qquad (8.26)$

Equation (8.26) is known as Lacey's regime equation and is of considerable use in evaluating flood discharges.

On the basis of the data of regime channels, Lacey also obtained

$$A f_1^2 = 140 U^5 \qquad (8.27)$$

i.e., $\quad Q f_1^2 = 140 U^6 \qquad (8.28)$

on substituting the value of f_1 from Eq. (8.24),

$$Q\left(\frac{5U^2}{2R}\right)^2 = 140 U^6$$

or $\quad \frac{25}{4}\,\frac{Q}{R^2} = 140 U^2$

or $\quad \frac{25}{4}\,\frac{Q P^2}{A^2} = 140 U^2$

$\sqrt{Q}$

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

i.e., $P = 4.733 \sqrt{Q} \approx 4.75 \sqrt{Q}$ (8.29)

Equation (8.29) with multiplying constant modified to 4.75 has been verified by a large amount of data and is very useful for fixing clear waterways for structures, such as bridges onrivers. Again, substituting the value of f_1 in Eq. (8.27),

$$A\left(\frac{5U^2}{2R}\right)^2 = 140U^5$$

$$\therefore \quad \frac{A}{R^2} = \frac{560}{25}U$$

or $\quad P = 22.4\ RU$

or $\quad 4.75\sqrt{Q} = 22.4RU$

or $\quad RU = 0.212\ Q^{1/2} \quad (8.30)$

For wide channels, RU equals the discharge per unit width. Hence,

$$q = 0.212\ Q^{1/2} \quad (8.31)$$

Equation (8.31) relates the discharge per unit width of a regime channel with the total discharge flowing in the channel.

On substituting the value of U from Eq. (8.24) in Eq. (8.30), one gets

$$R\sqrt{\frac{2}{5}f_1R} = 0.212\ Q^{1/2}$$

$$\therefore \quad R = 0.48\left(\frac{Q}{f_1}\right)^{1/3} \quad (8.32)$$

For wide channels, the hydraulic radius is almost equal to the depth of flow. Equation (8.32), therefore, gives the depth of scour below high flood level. Hence, Eq. (8.32) can be utilised to estimate the depth of flow in a river during flood. This information forms the basis for the determination of the levels of foundations, vertical cutoffs, and lengths of launching aprons of a structure constructed along or across a river.

On combining Eqs. (8.31) and (8.32),

$$R = 1.35\left(\frac{q^2}{f_1}\right)^{1/3} \quad (8.33)$$

Further, eliminating U from Eqs. (8.24) and (8.26), one can obtain a relationship for the slope of a regime channel. Thus,

$$\sqrt{\frac{2}{5}f_1R} = 10.8R^{2/3}\ S^{1/3}$$

$$\therefore \quad S = 0.0002\ \frac{f^{3/2}}{R^{1/2}} \quad (8.34)$$

On substituting the value of R from either Eq. (8.32) or Eq. (8.33), in Eq. (8.34), one obtains,

5/3

f

$S = 0.0003 \frac{1}{\quad}$

$Q^{1/6}$ (8.35)

$5/3$

and $S = 0.000178 \frac{1}{\quad}$ f (8.36)

$q^{1/3}$

Lacey's regime relations, Eqns. (8.23) to (8.36), are valid for regime channels and can be used suitably to design a regime channel for a given discharge and sediment size.

The following flow equation was also obtained by Lacey (11) :

$$U = \frac{1}{N_a} R^{3/4} S^{1/2} \qquad (8.37)$$

where, $N_a = 0.0225 f_1^{1/4}$ (8.38)

This means that the absolute roughness coefficient N_a is dependent only on the sediment size. On examining the data from various channels, the value of N_a was, however, not found to be constant. Lacey introduced the concept of 'shock' to explain the variation in N_a. He contended that a non-regime channel requires a larger slope (i.e., large value of N_a) to overcome what he termed as ' shock resistance' or the resistance due to bed irregularities. The shock resistance can, therefore, be considered similar to the form resistance of the bed undulations. This concept of Lacey leads one to conclude that a regime channel is free from shock. It is, however, known that the geometry of bed undulations can change even for the same sediment size and, hence, Lacey's contention that a regime channel is free from shock, is unacceptable (12).

Lacey's equations, commonly used for the design of alluvial channels, are summarised below :

$$f_1 = 1.76\sqrt{d} \qquad (8.25)$$

$$U = 10.8\, R^{2/3} S^{1/3} \qquad (8.26)$$

$$P = 4.75\sqrt{Q} \qquad (8.29)$$

$$R = 0.48\left(\frac{Q}{f}\right)^{1/3} \qquad (8.32)$$

$$S = 0.0003\,\frac{f_1^{5/3}}{Q^{1/6}} \qquad (8.35)$$

Comments on Regime Equations

The regime equations have been empirically derived using data of regime channels. These channels carried relatively less sediment load (approximately 500ppm). Thus, the equations can be expected to yield meaningful results only for the conditions in which the sediment load is of the same order as was being carried by the channels whose data have been used in obtaining the regime relations. The dimensions of a stable channel will be affected by the water discharge, the sediment characteristics and the amount of sediment load in the channel. The regime relations take into account only the water discharge and the sediment size but do not take into account the amount of sediment material being transported by the channels. Nevertheless, the regime equations do provide useful information which is very helpful in the design of unlined channels and other structures on alluvial rivers.

Example 8.6 Design a stable channel for carrying a discharge of 30 m^3/s using Lacey's method assuming silt factor equal to 1.0.

Solution: From Eq. (8.29),

$$P = 4.75\sqrt{Q} = 4.75\sqrt{30.0} = 26.02 \text{ m}$$

From Eq. (8.32),

$$R = 0.48(Q/f_1)^{1/3} = 0.48\left(\frac{30.0}{1.0}\right)^{1/3} = 1.49\ \text{m}$$

From Eq. (8.35),

$$S = 3 \times 10^{-4}\ f_1^{5/3}/Q^{1/6} = 3 \times 10^{-4}\ (1.0)^{5/3}/(30)^{1/6}$$

$$= 1.702 \times 10^{-4}$$

Form Eq. (8.26),

$$U = 10.8\ R^{2/3}\ S^{1/3} = 10.8(1.49)^{2/3}\ (1.702 \times 10^{-4})^{1/3}$$

$$= 0.781 \text{ m/s}$$

Assuming final side slope of the channel as 0.5H : 1V (generally observed field value),

$P = B + (\sqrt{5})h = 26.02$ m

$\therefore \quad B = 26.02 - 2.24h$

and $\quad A = Bh + \dfrac{h^2}{2} = PR = 26.02 \times 1.49 = 38.77 \text{ m}^2$

$\therefore \quad 26.02h - 2.24h^2 + 0.5h^2 = 38.77$

or $\quad 1.74h^2 - 26.02h + 38.77 = 0$

$\therefore \quad h = \dfrac{26.02 \pm \sqrt{(26.02)^2 - 4 \times 1.74 \times 38.77}}{2 \times 1.74}$

$= \dfrac{26.02 \pm 20.18}{3.48}$

= 13.28 m and 1.68 m

The value of h equal to 13.28 m gives negative B and is, therefore, not acceptable. Hence,

h = 1.68 m, and

B = 26.02 – 2.24 × 1.68

= 22.23 m

Example 8.7 An irrigation channel is to be designed for a discharge of 50 m^3/s adopting the available ground slope of 1.5×10^{-4}. The river bed material has a median size of 2.00 mm. Design the channel and recommend the size of coarser material to be excluded or ejected from the channel for its efficient functioning.

Solution: From Eqs. (8.25) and (8.35),

$f_1 = 1.76\sqrt{d}$

$= 1.76\sqrt{2}$

$= 2.49$

and $\quad S = 0.0003\ f_1^{5/3}/Q^{1/6}$

$= 0.0003\ (2.49)^{5/3}/(50)^{1/6}$

$= 7.15 \times 10^{-4}$

The computed slope is much large than the available ground slope of 1.5×10^{-4} which is to be adopted as the channel bed slope. Therefore, the median size of sediment which the channel would be able to carry can be determined by computing the new value of f_1 for $S = 1.5 \times 10^{-4}$ and the given discharge and then obtaining the value of d for this value of f_1 using Eq. (8.25). Thus,

$$1.5 \times 10^{-4} = 0.0003\ f_1^{5/3}/(50)^{1/6}$$

∴ $f_1 = 0.976$

and $d = (f_1/1.76)^2 = 0.30$ mm

Therefore, the material coarser than 0.30 mm will have to be removed for the efficient functioning of the channel. The hydraulic radius of this channel R is obtained from Eq. (8.32) :

$$R = 0.48 \left(\frac{50}{\;}\right)^{1/3}$$

∴ $R = 1.783$ m

Using Eq. (8.29)

$$P = 4.75\sqrt{50}$$
$$= 33.59 \text{ m}$$

∴ $B + \sqrt{5}\,h = 33.59$

or $B = 33.59 - 2.24h$

and $A = Bh + \frac{h^2}{2} = PR = 33.59 \times 1.783 = 59.89 \text{ m}^2$

or $33.59h - 2.24h^2 + 0.5h^2 = 59.89$

or $1.74h^2 - 33.59h + 59.89 = 0$

∴ $h = \frac{33.59 \pm \sqrt{(33.59)^2 - 4 \times 1.74 \times 59.83}}{2 \times 1.74}$

$= \frac{33.59 \pm 26.67}{3.84}$

$= 17.32$ m or 1.99 m

Obviously, h = 1.99 m as the other root of h would result in too narrow a channel section.

∴ $B = 33.59 - 2.24(1.99)$

$= 29.13$ m

8.5.2. Method of Design of Alluvial Channels IncludingSediment Load as a Variable

Experiments have revealed (13) that the stable width of a regime channel is practically independent of sediment load. Hence, one can use Lacey's equation [Eq. (8.29)] for stable perimeter P even when the sediment load is varying. If the bank soil is cohesive, the perimeter may be kept smaller than that given by Eq. (8.29). However, the sediment load is known to affect the regime slope of a channel and, hence, the sediment load should also be included inthe design of stable alluvial channels.

If the sediment load is moving mainly in the form of bed load, one can determine the unknowns R and S for given Q, q_B, d, and n using the Meyer-Peter's equation [Eq. (7.33)] andthe Manning's equation [Eq.(8.12)]

If the suspended sediment load is also considerable, then one may use the total load equation instead of Meyer-Peter's equation. Combining Engelund and Hansen equation [Eq. (7.37)] for sediment load,

$$f\phi_T = 0.4\tau_*^{5/2} \quad (8.39)$$

and their resistance equation (14),

$$U = 10.97\, d^{-3/4}\, h^{5/4}\, S^{9/8} \qquad (8.40)$$

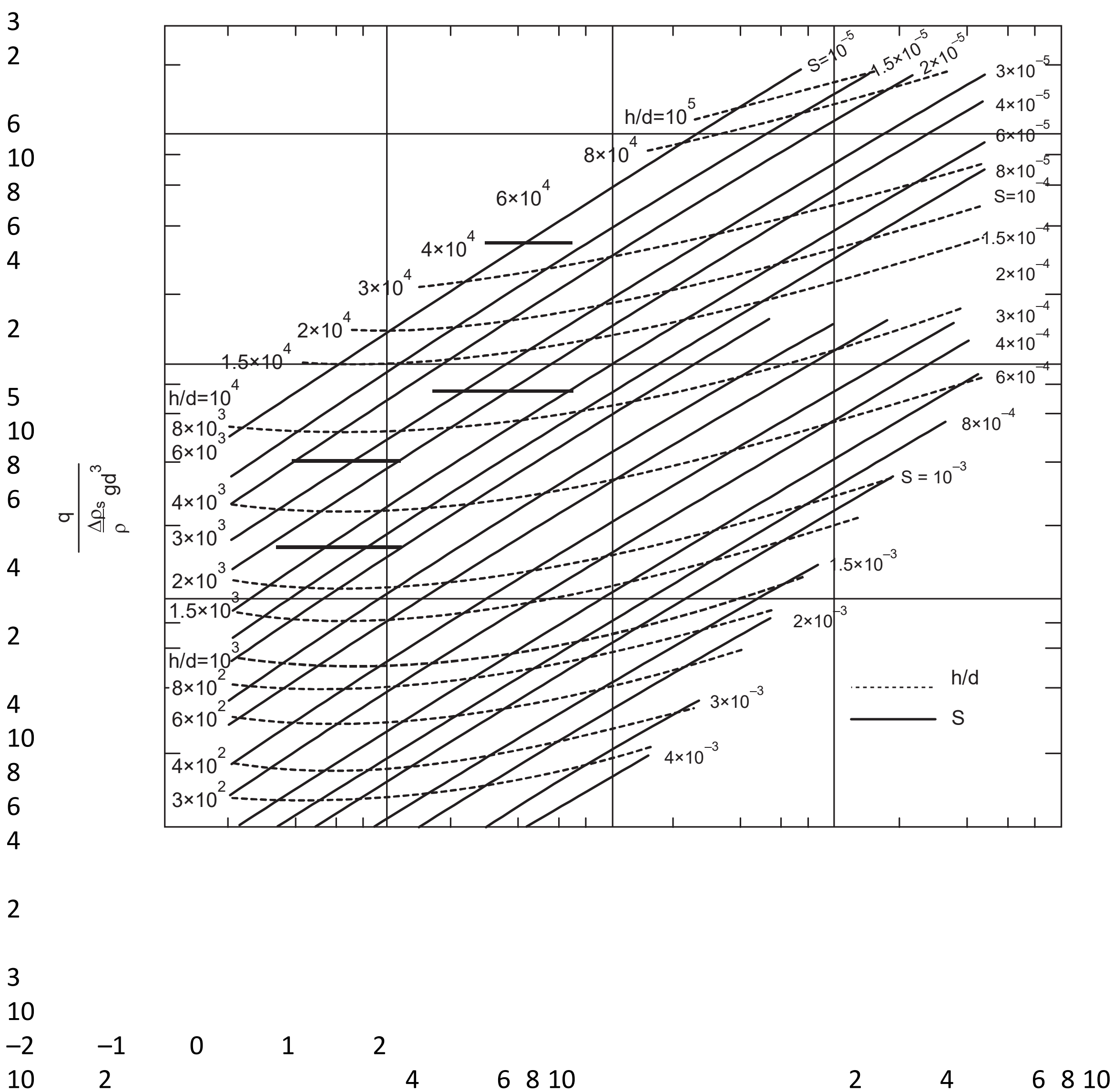
a design chart (Fig. 8.7) was prepared. This chart can be used to determine h and S, for knownvalues of

$$\Phi_T = \frac{q_T/\rho_s g}{\sqrt{\frac{\Delta\rho_s}{\rho} g d^3}} \quad \text{and} \quad \frac{q}{\sqrt{\frac{\Delta\rho_s}{\rho} g d^3}}$$

For the width of stable channel, Engelund and Hansen (14) suggested the use of the empirical relation

$$B = \frac{0.786 Q^{0.525}}{d^{0.316}} \qquad (8.41)$$

Equation (8.41) is valid for channels with sandy bed and banks (12). However, because of cohesive soils of India, the width of stable channels is smaller than that given by Eq. (8.41).



□T 2 4 2 4 6

6 8 8 10

10

Fig. 8.7 Engelund and Hansen's chart for stable channel design

Therefore, one can adopt a value of B which is slightly less than or equal to P obtained from Lacey's equation [Eq. (8.29)].

Equations (8.39) and (8.40) can be solved for h and S to yield the following explicit relations :

$$h = 0.17 \frac{q^{20/21}}{(g\phi_T)^{2/7}\, d^{3/7}\, (\Delta\rho_s/\rho)^{5/7}} \tag{8.42}$$

$$S = 8.4 \frac{q^{8/9}\, d^{2/3}}{h^2} \tag{8.43}$$

or
$$S = 4.12 \frac{(g\phi_T)^{4/7}\, d^{32/21}\, (\Delta\rho_s/\rho)^{10/7}}{q^{64/63}} \tag{8.44}$$

Equations (8.41), (8.42), and (8.43) or (8.44) will yield direct solutions for B, h and S, and avoid interpolation errors involved in the use of the design chart (Fig. 8.7).

In Eqs. (8.40) to (8.44), U is expressed in m/s, Q in m^3/s, q in m^2/s, q_T in N/m/s, and h, d, and B in m. It should be noted that Eq. (8.42) is valid for duned-bed conditions.

Example 8.8 Design a channel to carry a discharge of 30 m^3/s with sediment load concentration of 50 ppm by weight. The average grain size of the bed material is 0.3 mm.

Assume the cross-section of the channel as trapezoidal with side slopes $\frac{1}{2}$ H : 1V.

Solution: Using Eq. (8.29)

$$P = 4.75\sqrt{Q} = 4.75\sqrt{30} = 26.02 \text{ m}$$

Choose B to be slightly less than P, say B = 24.0 m.

$$Q_T = 50 \times 10^{-6} \times 9810 \times 30 = 14.71 \text{ N/s}$$

$$\therefore \quad q_T = \frac{14.71}{24.0} = 0.613 \text{ N/m/s}$$

and
$$q = \frac{30.0}{24.0} = 1.25 \text{ m}^2/\text{s}$$

Now
$$\phi_T = \frac{0.613/(2.65 \times 9810)}{\sqrt{1.65 \times 9.81 \times (0.3 \times 10^{-3})^3}} = 1.128$$

and
$$\frac{q}{\sqrt{\left(\frac{\Delta\rho_s}{\rho}\right) g d^3}} = \frac{1.25}{\sqrt{1.65 \times 9.81 \times (0.3 \times 10^{-3})^3}} = 59793.2$$

From Fig. 8.7,

$$S = 1.1 \times 10^{-4}$$

and
$$\frac{h}{d} = 7.2 \times 10^3$$

$$\therefore \quad h = 7.2 \times 10^3 \times (0.3 \times 10^{-3}) = 2.16 \text{ m}$$

Alternatively, using Eqs. (8.42) and (8.43),

$$h = \frac{0.17 \times (1.25)^{20/21}}{(9.81 \times 1.128)^{2/7} (0.3 \times 10^{-3})^{3/7} (1.65)^{5/7}}$$

$= 2.38$ m

$$S = \frac{8.4 \times (1.25)^{8/9} \times (0.3 \times 10^{-3})^{2/3}}{(2.38)^2}$$

$= 8.1 \times 10^{-3}$

Example 8.9 Solve Example 8.8 considering the entire sediment load as bed load.

Solution: $Q_B = 50 \times 10^{-6} \times 9810 \times 30 = 14.71$ N/s

Silt factor, $f_1 = 1.76\sqrt{d} = 1.76\sqrt{0.3} = 0.964$

Since the value of Manning's n is 0.0225 for $f_1 = 1.0$, one can take n = 0.022 for $f_1 = 0.964$

Using Eq. (8.29),

$P = 4.75\sqrt{30.0} = 26.02$ m

Let $B = 24.0$ m

∴ $q_B = \frac{14.71}{24.0} = 0.613$ N/m/s

Hence,

$$\Phi_B = \frac{q_B/(\rho_s g)}{\sqrt{\frac{\Delta\rho_s}{\rho} g d^3}} = \frac{0.613/(2650 \times 9.81)}{\sqrt{1.65 \times 9.81 \times (0.3 \times 10^{-3})^3}} = 1.128$$

and from Eq. (7.10)

$$n_s = \frac{d^{1/6}}{25.6}$$

$$= \frac{(0.3 \times 10^{-3})^{1/6}}{25.6}$$

$= 0.01$

∴ $$\tau_* = \left(\frac{n_s}{n}\right)^{3/2} \frac{RS}{\Delta_s d} = \left(\frac{0.01}{0.022}\right)^{3/2} \frac{RS}{1.65 \times (0.3 \times 10^{-3})}$$

$= 619.10$ RS

∴ Using Meyer-Peter's equation [Eq. (7.33)],

$$\Phi_B = 8.0(\tau_* - 0.047)^{3/2}$$

or $1.128 = 8.0(619.10RS - 0.047)^{3/2}$

∴ $RS = 5.135 \times 10^{-4}$

Now using the Manning's equation,

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

$$30 = \frac{1}{0.022}(26.02 \times R^{5/3} S^{1/2})$$

$\therefore \quad R^{5/3} S^{1/2} = 0.0254$ or $R^{7/6}(5.135 \times 10^{-4})^{1/2} = 0.0254$

$\therefore \quad R = 1.103$ m

and $\quad S = 4.66 \times 10^{-4}$

$$A = Bh + \frac{h^2}{2} = 24h + \frac{h^2}{2} = PR = 26.02 \times 1.103$$

or $\quad h^2 + 48h - 57.4 = 0$

$$h = \frac{-48 \pm \sqrt{(48)^2 + 4 \times 57.4}}{2}$$

h = 1.167 m

$P = B + \sqrt{5}\,h$

$= 24 + \sqrt{5} \times 1.67$

= 26.61 m

which is close to Lacey's perimeter (= 26.02 m)Hence,

B = 24.0 m h = 1.167 m and $S = 4.66 \times 10^{-4}$

PROCEDURE FOR DESIGN OF IRRIGATION CHANNELS

A canal system includes the main canal, branches, distributaries, and watercourses. Main canals, branches and distributaries are owned, constructed, controlled, and maintained by the government. Watercourses are owned, constructed, controlled, and maintained by the irrigators. Sometimes, the watercourses are constructed by the government agencies also. Their maintenance is, however, the responsibility of irrigators.

The main canal takes its supply from the source directly. The source may be either a river or a reservoir. There may be several branch canals and/or distributaries taking suppliesfrom the main canal at various points along the main canal. A branch channel, similarly, feeds distributaries which take off at suitable locations along the branch channel. A distributary feeds watercourses which take water directly to the field. The discharge and size of irrigationchannels progressively decrease from the main canal to the watercourses.

Layout and Discharge Requirements of Irrigation Channels

The layout of irrigation channels should ensure equitable distribution of water with minimum expenditure. It is, obviously, advantageous to align all irrigation channels on the watershed. This will ensure least expense on cross-drainage structures and cause gravity flow on either side of the channel. In hilly regions, the channels are aligned along the contours. Alignment of irrigation channels should be such that there are very few curves. However, if the channel curves are unavoidable, the radius of curvature of these curves should be as large as possible. For large channels, the minimum radius of curvature should be about 60 times the bed width of the channel. In case of smaller channels, the minimum radius of curvature of the curves should be about 45 times the bed width of the channel.

The maximum discharge to be carried by an irrigation channel at its head to satisfy the irrigation requirements for its command area under the worst conditions during any part of a year is said to be the designed full supply discharge (or capacity) of the channel. The water level in the channel when it is carrying its full supply discharge is termed full supply level (FSL) and the corresponding depth of flow is called full supply depth (FSD).

After fixing the layout of all irrigation channels (i.e., canal system) on a map, the total command area proposed to be irrigated by each of the channels of the canal system is estimated.Knowing the crop requirements and intensity of irrigation, and allowing for seepage and evaporation loses, the amount of the discharge requirement at various points of the canal system are determined. Obviously, this requirement will progressively increase from the tail end of a channel to its head reach. Accordingly, the size (i.e., width and depth) of a channel will progressively decrease form its head reach to its tail end. The size of a watercourse is, however, kept uniform.

Longitudinal Section of Irrigation Channels

Longitudinal Slope

An irrigation channel flowing in an alluvium can be designed by any of the methods discussedin Secs. 8.4 and 8.5. The slope of the channel would thus be known. However, it is convenientto choose a slope close to Lacey's regime slope [Eq. (8.35)] for the first trial design. This may need modification subsequently. The slope of an irrigation channel is governed by the countryslope too. If the ground slope is more than Lacey's regime slope (or the slope satisfying the critical velocity ratio and bed width-depth ratio, or that calculated on the basis of other methods), then the slope of the channel bed is kept as the required slope (i.e., the calculated one) and extra fall in the ground level is compensated for by providing vertical falls in the channel. Onthe other hand, if the required bed slope is more than the available ground slope, the maximum allowed by the ground slope is provided. In such cases, it is advisable to prevent the entry of coarse sediment at the head of the channel (see Example 8.7). Usually, ground slope is steeperthan the desired bed slope and falls are invariably provided along the channels. The slope of reservoir-fed channels, carrying water with practically no sediment, may be determined by the consideration of the maximum permissible velocity.

For convenience, the slope is kept in multiples of 2.5 cm/km. The advantage of such values is that the drop in the bed level for every 200 m would be 0.5 cm which can be easilyread on a levelling staff.

Full Supply Level (or Full Supply Line)

Since the channel has been aligned on the watershed, the available transverse slopes would besufficient for flow irrigation provided that the full supply level (FSL) is slightly above the ground level. The full supply level of a channel should, therefore, be about 10-30 cm higher than the ground level for most of its length so that most of the command area of the channel is irrigated by gravity flow. It is not advisable to keep the FSL higher than the isolated high patches of land existing in the command area. These may be irrigated either by lifting water from the watercourses or from the upstream outlets, if feasible. Too high an FSL would result in uneconomical channel cross-section of higher banks. Besides, the available higher head would increase the seepage loss, wasteful use of water by the farmers, and the chances of waterlogging in the area.

The FSL of the offtaking channel at its head should always be kept at least 15 cm lower than the water level of the parent (or supply) channel. This provision is made to account for the head loss in the regulator and for the possibility of the offtaking channel bed getting silted up in its head reaches.

Canal Falls

When the desired bed slope is flatter than the ground slope, as is usually the case, a canal fall has to be provided well before the channel bed comes into filling and the FSL becomes too much above the ground level. The location of a fall depends on a number of factors. It may becombined with a regulator or a bridge for economic benefits.

The alignment of another smaller offtaking channel may require that the fall be provided downstream of the head of the offtaking canal. This will ensure that the FSL of the offtaking channel is sufficiently high to irrigate its command area by flow irrigation. One may provide either a larger number of smaller falls or a smaller number of larger falls. Relative economy ofthese two possible alternatives should be worked out and the location of falls will be, accordingly, decided. The distributaries and watercourses offtaking from the upstream of a fall may easilyirrigate some areas downstream of the fall. As such the FSL in the channel may be allowed to remain below ground level for about 200-400 m downstream of a fall.

Balanced Earthwork

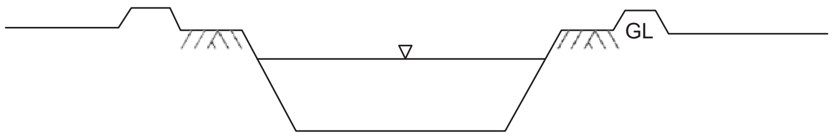
The longitudinal section of a channel should be such that it results in balanced earthwork which means that the amount of excavated soil is fully utilised in fillings. This will minimise the necessity of borrowpits (from where the extra earth needed for filling will be taken) andspoil banks (where the extra earth from excavation will be deposited). The earthwork is paidfor on the basis of either the excavation or the filling in embankments, whichever is more. A channel in balanced earthwork will, therefore, reduce the cost of the project. For the small length of a channel in the vicinity of a fall there is always an unbalanced earthwork which should be kept minimum for obvious economic reasons.

Evaporation and Seepage Losses in Canals

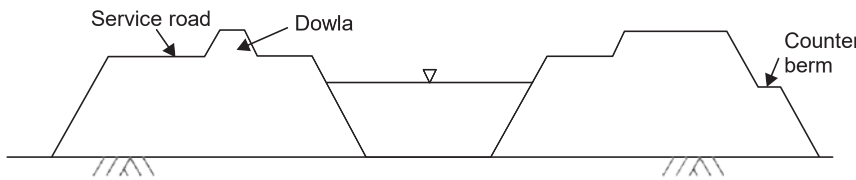
When water flows in an alluvial channel, some water is lost due to evaporation from the water surface and seepage through the bed and banks of the channel. These losses are termed transit losses or canal losses. The transit losses are calculated at the rate of about 3 m^3/s per million square metre of the exposed water surface area. These losses may be of the order 10 to 40 per cent of the discharge at the head of a channel in case of large channels.

Cross-Section of Irrigation Channels

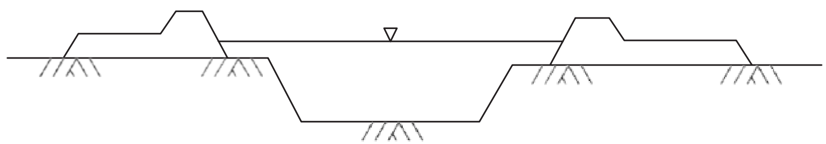
An irrigation channel may be in cutting or in filling or in partly cutting and partly filling. When the ground level is higher than the full supply level of the channel, the channel is said to be in cutting [Fig. 8.8 (a)]. If the channel bed is at or higher than the surrounding ground level, the channel is in filling, [Fig. 8.8(b)]. When the ground level is in between the supply level andthe bed level of the channel, the channel is partly in cutting and partly in filling [Fig. 8.8 (c)]. A channel section partly in cutting and partly in filling results in balanced earthwork and is preferred. Various components of cross-section of an irrigation channel have been discussed in the following paragraphs.



Channel in cutting



Channel in filling



Channel partly in cutting and partly in filling

Fig. 8.8 Typical cross-sections of irrigation channels

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 earthfill should not behave differently from an earthfill compacted sufficiently and having the same characteristics as those of the natural earthfill. 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, as recommended by the Central Water Commission of India, are given in Table 8.6.

Table 8.6 Side slopes for unlined channel

Type of soil	Side slopes (H : V)

Loose sand to average sandy soil Sandy loam and black cotton soil Gravel Murum or hard soil Rock	1.5 : 1 to 2 : 1 (in cutting) 2 : 1 to 3 : 1 (in filling) : 1 to 1.5 : 1 (in cutting) : 1 (in filling)1 : 1 to 2 : 1 0.75 : 1 to 1.5 : 1 0.25 : 1 to 0.5 : 1

Berms

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 to facilitate drainage.

Because of irregular changes in the ground level and regular changes in the channel bed level, the depth of cutting d_1 and the depth of filling, $(d_2 - d_1)$ (Fig. 8.9) would vary. This will cause variation in the horizontal distance between the bed and top of the bank (i.e., betweenX and Y). If r_1 (H) : 1(V) is the side slope in cutting and r_2(H) : 1(V) 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 Fig. 8.9, this distance is equal to $r_1 d_1 + (r_2 - r_1) d_1 + r_2(d_2 - d_1)$ which equals $r_2 d_2$. 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., $r_2 d_2$) also remains constant if the bermwidth is equal to $(r_2 - r_1)d_1$.

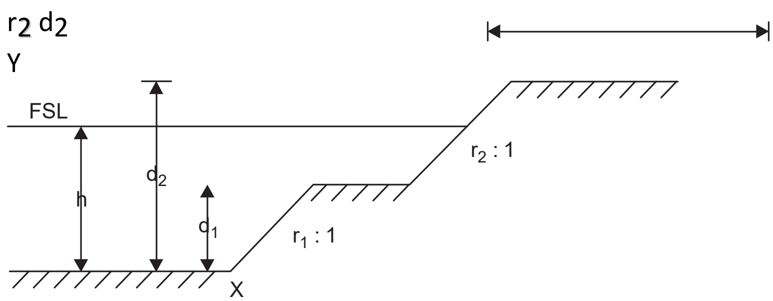


Fig. 8.9 Berm

For usual side slopes of 1.5 (H) : 1 (V) (in filling) and 1(H) : 1(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 berms, an impervious lining is formed on the banks. This helps in reduction of seepage losses. In addition, the berms protect 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 sectionswhich are in deep cutting.

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, land slides, 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 inlarge channels. For channels of intermediate size, freeboard is sometimes estimated by adding 0.3 m to one quarter of the flow depth.

The Central Water Commission of India (CWC) has recommended the value of freeboard as given in Table 8.7.

Table 8.7 Freeboard of irrigation channels as suggested by CWC

Discharge (m^3/s)	up to 0.7	0.7 to 1.4	1.4 to 8.5	over 8.5
Freeboard (m)	0.46	0.61	0.76	0.92

Alternatively, Table 8.8 may be used for the estimation of freeboard.

Table 8.8 Freeboard in irrigation channels (15)

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

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

$$F = \sqrt{CD}$$

where, C is a constant varying form 0.45 (for discharges up to 0.07 m^3/s) to 0.76 (for discharges greater than 85 m^3/s) and D is the water depth in metre.

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. This has been discussed in Chapter 15. 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 very from 4H : 1V for relatively impermeable material (such as ordinary loam soil) to 10H : 1V for porous sand andgravel. 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 Fig. 8.8(b), is provided. Alternatively, the outer slope of the bank is flattened.

The Central Water Commission has recommended bank widths as given in Table 8.9.

Table 8.9 Bank widths in irrigation channels

Discharge (m^3/s)	Top width of bank (m)
less than 0.28	0.92
0.28 to 1.4	1.22
1.4 to 4.2	1.50
4.2 to 10.0	1.83
10.0 to 14.0	2.44
14.0 to 28.0	3.66
28.0 to 140.0	4.58

Service Road and 'Dowla'

For proper maintenance and inspection of irrigation channels, service roads are provided on both sides of main canals and major branches. In the case of smaller branches and distributaries, a service road on only one side (usually, the left bank) is provided. The inspection road on main canals is usually about 6.0 m wide and should not be smaller than 5.0 m wide. The canal roads are generally unsurfaced but made motorable by using compacted and dressed earth material. If the canal road is to be used for other purposes as well, it should be metalled and surfaced.

If a road is constructed on the ground along a banked canal, the canal is not visible while driving along the road. As such, the vehicle has to be stopped frequently at places to have a look at the canal. Thus, proper inspection of the canal is difficult. This problem can be avoided if the road and the bank are combined together, [Fig. 8.8(b)] and a 'dowla' of about 0.5

m height and about 0.5 m top width is provided for safety reasons. The level of the road is kept about 0.3 to 0.75 m above full supply level of the canal.

Schedule of Area Statistics and Channel Dimensions

The calculations for the design of an irrigation channel are usually carried out in tabular form. The table containing these calculated values is called the 'schedule of area statistics and channel dimensions'. The calculations start from the tail end and the design is usually carried out at every kilometre of the channel downstream of the head of the channel. Intermediate sections are designed only in special circumstances such as a large reduction in discharge due to an offtaking channel in between the adjacent two sections. Example 8.10 illustrates the procedure for the preparation of schedule of area statistics and channel dimensions for an irrigation channel.

Example 8.10 Design the first 5 km of a distributary channel which takes off from a branch the bed of which is at 224.0 m and the water depth in the branch is 2.0 m. The channel is to be designed for Rabi irrigation of intensity 30 per cent. Outlet discharge factor is 1800 hectares/m^3/s.

CCA = 70 per cent of GCA

Evaporation and seepage losses = 2.5 $m^3/s/10^6$ sq.mCritical velocity ratio = 1.05

Manning's n = 0.023

Silt factor, f_1 = 1.0

Losses downstream of 5 km section = 0.28 m^3/sField application efficiency = 65% Provide slope close of Lacey's slope

Distance from head of canal in km	GCA (hectares)
0	20,800
1	18,850
2	15,990
3	12,610
4	10,725
5	9,230

Ground Levels

Dist. (km)	RL (m)	Dist. (km)	RL(m)
0.0	225.00	2.6	223.20
0.2	225.00	2.8	223.05
0.4	225.15	3.0	222.90
0.6	225.10	3.2	222.80
0.8	225.05	3.4	222.65
1.0	224.95	3.6	222.50
1.2	224.25	3.8	222.55
1.4	224.10	4.0	222.40
1.6	224.05	4.2	222.30
1.8	224.00	4.4	222.25
2.0	223.95	4.6	222.20
2.2	223.35	4.8	222.15
2.4	223.20	5.0	222.00

Solution: Calculations for this problem have been shown in Table 8.10. Columns 1, 2 and 7 are as per the given data. CCA (col. 3) is obtained by multiplying GCA with given factor of 0.7. CCA (col. 3) is multiplied by the intensity of irrigation (30 per cent, i.e., 0.3) to obtain the area to be irrigated for Rabi crop (col. 4). If intensities of irrigation for Kharif crops and sugarcane crop (or some other major crop) are also known, then corresponding areas to be irrigated for Kharif crop (col. 5) and sugarcane or other crop (col. 6) can also be similarly obtained. For known outlet discharge factors for Rabi, Kharif and Sugarcane (or other) crops, corresponding outlet discharges can be calculated by dividing the area to be irrigated with the corresponding outlet discharge factor and also the field application efficiency. Maximum of these outlet discharges is entered in col. 10 of Table 8.10.

Now a step-by-step method is followed from the tail end to determine channel dimensions in different reaches.

To the outlet discharge below 5.0 km (col. 10) is added the given losses downstream of 5 km section (i.e., 0.28 m^3/s) entered in col. 12 to obtain the total discharge (col. 13).

Irrigation channels are designed for a discharge which is 10 per cent more than the total discharge required so that the channels can carry increased supplies in times of keen demand. The design discharge has been entered in col. 14.

Channel dimensions (i.e., S, B, and h) can now be computed using a suitable method. Bed slope has been obtained from Lacey's equation [Eq. (8.35)]. This value of slope is modified to the nearest multiple of 2.5 and entered in col. 15. The method proposed by Ranga Raju and Misri (8) has been used to obtain B and h. Alternatively, Kennedy's method of trial or Lacey's equations can be used to obtain the channel dimensions.

In the present problem, since the adopted value of p is different from the value of p obtained from Fig. 8.7, the values of velocity obtained from the Manning's equation [Eq. (8.12)], and the Kennedy's equation [(Eq. 8.9)] should be compared. If the two values differ considerably, the channel dimensions should be revised suitably. For the present problem, the difference varies from 1.5 per cent to 6.5 per cent only for the chosen channel dimensions.

Having obtained the channel dimensions at the tail end (in this case at 5 km section), the losses in the reach between km 4 and km 5 are estimated. The water surface width at the km 5 section is 4.65 m (side slope of channel is 1/2H : 1V). Assuming average water surface width in the reach (between km 4.0 and km 5.0) as 5.0 m, the loss in the reach is equal to $5 \times 1000 \times 2.5 \times 10^{-6} = 0.0125$ m^3/s (col. 11) which is added to the losses downstream of 5 km (0.28 m^3/s) to obtain total losses downstream of 4 km which comes to 0.2925 m^3/s (col. 12).

Channel dimensions can now be estimated for the reach between km 4.0 and km 5.0 as explained for the reach downstream of km 5.0. The average water surface width for the reach between km 4.0 and km 5.0 works out to $(1/2)(4.9 + 4.65) = 4.775$ m and the corresponding reach loss is 0.012 m^3/s which is not much different from the assumed reach loss of 0.0125 m^3/s. If the difference is large, the computations of channel dimensions may have to be revised.

Following the above procedure the channel dimensions up to the head of canal are determined. The computations have been shown in Table 8.10.

Longitudinal section is shown plotted in Fig. 8.10. Also shown in this figure are two falls which should be located keeping in view the requirements and guidelines discussed in Sec. 8.6.2.

BORROW PITS, SPOIL BANKS, AND LAND WIDTHFOR IRRIGATION CHANNEL

Borrow Pits and Spoil Banks

Although it is advisable to keep the channel in balanced earth work, it is generally not possible to do so. If the amount of earth required for filling is more than the amount of excavated earth, then the excess requirement of filling is met by digging from suitably selected areas known as borrow pits, Fig. 8.11 (a).

If unavoidable, borrow pits should be made in the bed or berms of the channel. These pits will silt up after sometime when water has flowed in the canal. The depth of the borrow pits is kept less than 1 m and the width is limited to half the bed width. The borrow pits are located centrally in the channel bed and are spaced such that the distance between adjacent borrow pits is at least half the length of the borrow pits. Borrow pits may be similarly locatedin wide berms of a channel.

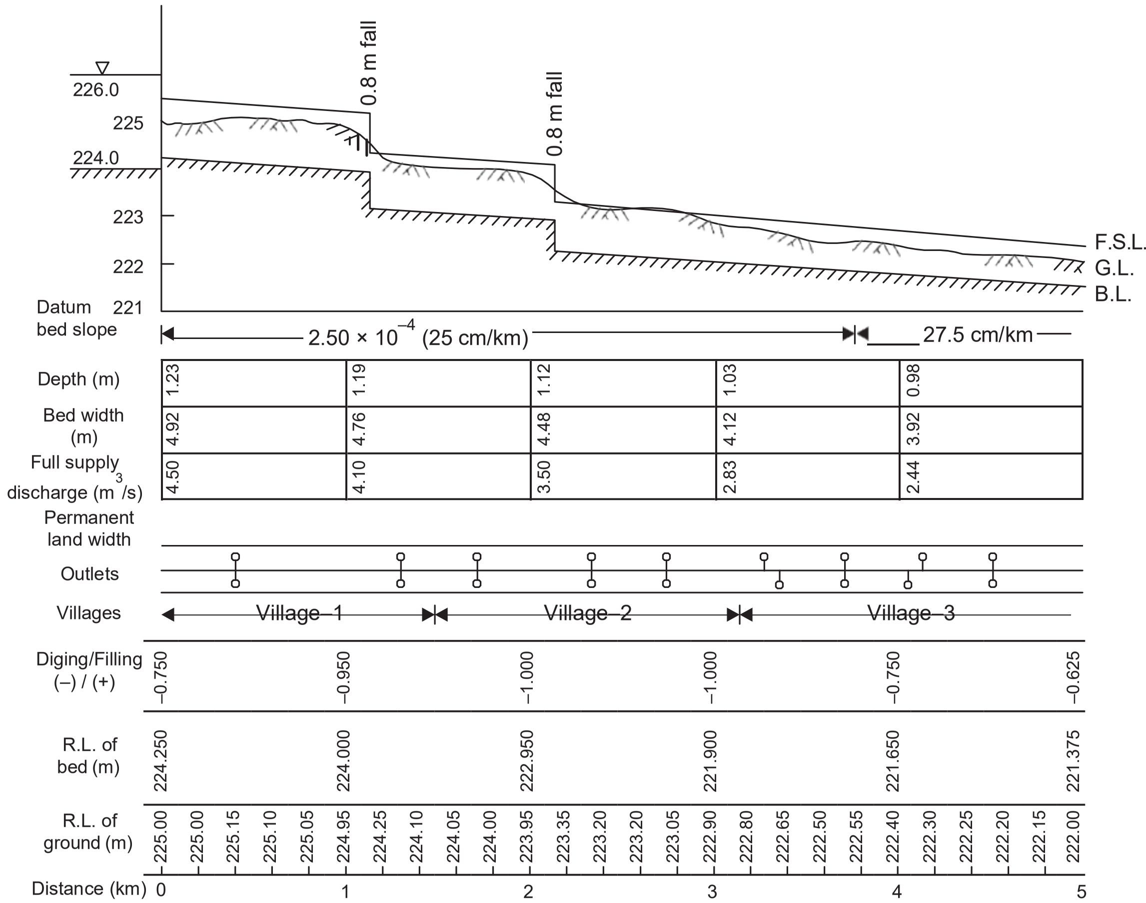
If the material from internal borrow pits is not sufficient to meet the requirement then extra material is taken from external borrow pits which should be about 5 to 10 m away from the toe of the canal bank. These should not be deeper than 0.3 m and should always be connected to a drain.

Table 8.10 Schedule of area statistics and channel dimensions (Example 8.10)

Distance from head of canal	GCA	CCA	Area to be irrigated			Outlet discharge factor		
			Rabi	Kharif	Sugarcane or other major crop	Rabi	Kharif	Sugarcane or other crop
km	Ha	ha	ha	ha	ha	ha/m^3/s	ha/m^3/s	ha/m^3/s
1	2	3	4	5	6	7	8	9
0	20800	14560	4368	–	–	1800	–	–
1	18850	13195	3958.5	–	–	1800	–	–
2	15990	11193	3357.9	–	–	1800	–	–
3	12610	8827	2648.1	–	–	1800	–	–
4	10725	7507.5	2252.25	–	–	1800	–	–
5	9230	6461	1938.3	–	–	1800	–	–

Table 8.10 (continued)

Distance from head of canal	Outlet discharge	losses in reach	Total losses	Total discharge	Design discharge	Channel Dimensions					
						Bed slope	$\frac{SQ^{0.02}}{(n^2m^2)}$	p from Fig. 8.6	Adopted p = B/h	h from Eq.(8.16)	Channel width B = ph
km	m^3/s	m^3/s	m^3/s	m^3/s	m^3/s					m	M
	10	11	12	13	14	15	16	17	18	19	20
0	3.733	0.0165	0.3540	4.0870	4.50	2.50×10^{-4}	0.442	5.0	4.0	1.23	4.92
1	3.383	0.0160	0.3375	3.7205	4.10	2.50×10^{-4}	0.441	5.0	4.0	1.19	4.76
2	2.870	0.0150	0.3215	3.1915	3.50	2.50×10^{-4}	0.440	5.0	4.0	1.12	4.48
3	2.263	0.0140	0.3065	2.5695	2.83	2.50×10^{-4}	0.438	5.0	4.0	1.03	4.12
4	1.925	0.0125	0.2925	2.2175	2.44	2.75×10^{-4}	0.480	4.0	4.0	0.98	3.92
5	1.657	–	0.2800	1.9370	2.13	2.75×10^{-4}	0.479	4.0	4.0	0.93	3.72



Distance (km)	0	1	2	3	4	5
Depth (m)	1.23	1.19	1.12	1.03	0.98	
Bed width (m)	4.92	4.76	4.48	4.12	3.92	
Full supply discharge (m^3/s)	4.50	4.10	3.50	2.83	2.44	
Diging/Filling (–) / (+)	–0.750	–0.950	–1.000	–1.000	–0.750	–0.625
R.L. of bed (m)	224.250	224.000	222.950	221.900	221.650	221.375

R.L. of ground (m)
225.00
225.00
225.15
225.10
225.05
224.95
224.25
224.10
224.05
224.00
223.95
223.35
223.20
223.20
223.05
222.90
222.80
222.65
222.50
222.55
222.40
222.30
222.25
222.20
222.15
222.00

Fig. 8.10 Longitudinal section of distributary channel (Example 8.10)

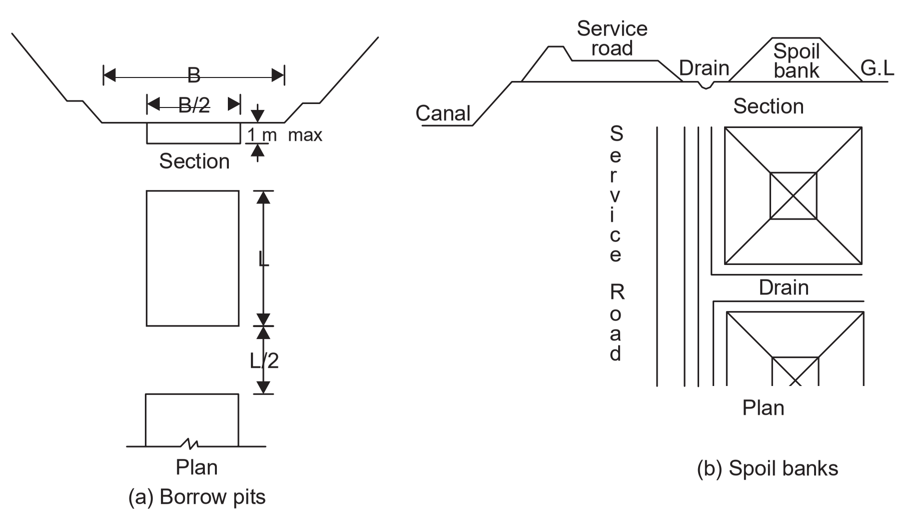


Fig. 8.11 Borrow pits and spoil banks

2.13 3.72 0.93

On the other hand, if the excavated earth exceeds the requirement of earth material for the construction of banks and service roads, the excess earth has to be suitably disposed of. If the excess earth is not much, it can be used to widen or raise the canal banks. If the quantity of excess earth is much larger, then it is utilised to fill up local depressions in the area or deposited in spoil banks, Fig. 8.11 (b), on one or both sides of the channel. The section of spoil banks depends on the cost of the land and labour. These should be provided with good cross slopes on all sides for ensuring proper drainage. The spoil banks should be discontinuous to allow cross drainage between them.

Land Width

Land width required for the construction of a channel includes the permanent and temporary land. The permanent land extends a little beyond the outer toe of the canal banks (or the drain, if provided) on either side. If trees are to be planted along the canals, then the land acquired for this purpose should also be included in the permanent land. Planting of trees adjacent to good culturable land should be avoided as the shade of the trees would affect the plants.

In addition to the permanent land, some land along the channel is required during construction for the storage of materials and equipment, and other purposes related to the construction work. This temporary land is returned to the concerned owners after use withdue compensation.

SEDIMENT DISTRIBUTION IN AN ALLUVIAL CHANNEL

The sediment distribution along a vertical is given by the Rouse equation [Eq. (7.34)]. In the channels of north India, the concentration of sediment discharge by weight near the water surface is found to be 40 to 70 per cent of the average value which is generally found at about

0.6 times depth below the water surface. The percentage of coarse sediment continuously increases towards the bed.

Lateral sediment distribution is such that there is generally greater concentration of sediment near the banks than at the centre. This is due to cross currents in a channel (Fig. 8.12).

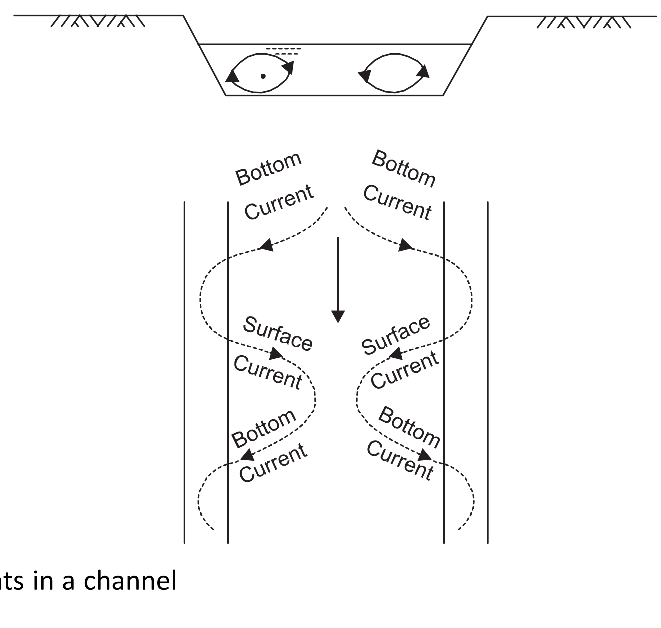


Fig. 8.12 Cross currents in a channel

Surface water flowing faster than the lower layer water, tends to topple over the slower moving water and goes to the bed of the channel. At the bed, this water is deflected towards the bank. This cross current at the bed (where the concentration of coarse sediment is the greatest) from the centre towards the bank pushes the coarse sediment towards the banks. The return current is toward the centre and upwards and, hence, the lifting up of the sediment is opposed by gravity.

The tendency of silting near the banks due to cross currents can be reduced by accelerating the flow in the bank region by some means such as pitching the bank slope.

Flow along the bends in the channel is such that there is heading up of water near the outer bank due to which there is a cross current of bottom water towards the inner bank. This leads to the deposition of sediment near the inner bank. Thus, there is shallow depth on the inner bank of the bend and greater depth on the outer bank of the bend. A channel should, therefore, offtake from the outer bank if it has to draw relatively clear water.

In a bell-mouthed converging channel, the sediment concentration is maximum near the central part of the channel. An offtaking channel from such a reach would, therefore, draw comparatively less sediment.

SILTING AND BERMING OF CHANNELS

In the head reaches of a distributary channel there may exist a tendency of silting up due to one or more of the following reasons :

Defective Head Regulator : A defective head regulator may make the offtaking chan- nel draw a higher percentage of coarser sediment. This increases the silt factor and sediment load of the channel and leads to the silting of the channel.

Non-regime Section : If the channel section is not able to carry its sediment load, the excess sediment is deposited on the channel bed. In due course, this deposition of sediment increases the channel slope so that the channel becomes capable of carry-ing the sediment which enters the channel. The channel may, then, continue in tem- porary equilibrium of 'initial regime'.

Inadequate Slope : If the channel slope is less than the regime slope, the sediment will deposit on the channel bed so as to increase the slope to the regime slope. In cases where the ground slope is less than the regime slope, the entry of coarse sedi- ment into the channel should be minimised so that the silt factor of the channel sediment reduces.

Fluctuation in Supply : When the channel is running with low supplies for a long period, it gets silted up in the head reaches.

Defective Outlets : If the outlets do not draw their share of sediment in proportion with the water discharge, the channel will have a tendency to silt up. This is due to the fact that downstream of the outlet, the parent channel has to carry relatively more sediment with reduced water discharge.

In the tail reaches of a channel, the discharge and velocity are both reduced. The velocities near the sides of the channel are very low and, hence, the silting takes place near the sides. The grass on the channel sides also catches the sediment. This is termed 'berming' of the channel and should be distinguished from the 'berms' which are deliberately provided in the channel section. The berming results in reduction of the channel section and, hence, rise in water levels. To improve the channel condition in the tail reaches, sometimes berm cutting may have to be carried out.

3.1.7 Farm Drainage and Plaaning Design

Irrigation and Drainage System

Irrigation and drainage, artificial application of water to land and artificial removal of excess water from land, respectively. Some land requires irrigation or drainage before it is possible to use it for any agricultural production; other land profits from either practice to increase production. Some land, of course, does not need either. Although either practice may be, and both often are, used for nonagricultural purposes to improve the environment, this article is limited to their application to agriculture. Irrigation and drainage improvements are not necessarily mutually exclusive. Often both may be required together to assure sustained, high-level production of crops.

Modern irrigation system planning and construction
The first consideration in planning an irrigation project is developing a water supply. Water supplies may be classified as surface or subsurface. Though both surface and subsurface water come from precipitation such as rain or snow, it is far more difficult to determine the origin of subsurface water.

In planning a surface water supply, extensive studies must be made of the flow in the stream or river that will be used. If the stream flow has been measured regularly over a long period, including times of drought and flood, the studies are greatly simplified. From stream flow data, determinations can be made of the minimum, maximum, average daily, and average monthly flows; the size of dams, spillways, and downstream channel; and the seasonal and carry-over storage needed. If adequate stream flow data are not available, the stream flow may be estimated from rain and snow data, or from flow data from nearby streams that have similar climatic and physiographic conditions.

The quality, as well as the quantity, of surface water is a factor. The two most important considerations are the amount of silt carried and the kind and amount of salts dissolved in the water. If the silt content is high, sediment will be deposited in the reservoir, increasing maintenance costs and decreasing useful life periods. If the salt concentration is high, it may damage crops or accumulate in the soil and eventually render it unproductive.

Subsurface sources of water must be as carefully investigated as surface sources. In general, less is known about subsurface supplies of water than about surface supplies, so, therefore, subsurface supplies are harder to investigate. Engineers planning a project need to know the extent of the basic geological source of water (the aquifer), as well as the amount the water level is lowered by pumping and the rate of recharge of the aquifer.
Often the only way for the engineer to obtain these data reliably is to drill test wells and make on-site measurements. Ideally, a project is planned so as not to use more subsurface water than is recharged. Otherwise, the water is said to be “mined,” meaning that it is being used up as a natural resource and its use is considered unsustainable.

Two sources of water not often thought of by the layperson are dew and sewage or wastewater. In certain parts of the world, Israel and part of Australia, for example, where atmospheric conditions are right, sufficient dew may be trapped at night to provide water for irrigation. Elsewhere the supply of wastewater from some industries and municipalities is sufficient to irrigate relatively small acreages. Recently, due to greater emphasis on purer water in streams, there has been increased interest in this latter practice.

In some countries (Egypt for example) sewage is a valuable source of water. In others, such as the United States,irrigation is looked upon as a means of disposing of sewer water as a final step in the wastewater treatment process. Unless the water contains unusual chemical salts, such as sodium, it is generally of satisfactory quality for agricultural irrigation. Where the practice is used primarily as a means of disposal, large areas are involved and the choice of crop is critical. Usually only grass or trees can withstand the year-round applications.

Before a water supply can be assured, the right to it must be determined. Countries and states have widely varying laws and customs that determine ownership of water. If the development of a water supply is for a single purpose, then the determination of ownership may be relatively simple; but if the development is multipurpose, asmost modern developments are, ownership may be difficult to determine, and agreements must be worked out among countries, states, municipalities, and private owners.

The area that can be irrigated by a water supply depends on the weather, the type of crop grown, and the soil. Numerous methods have been developed to evaluate these factors and predict average annual volume of rainfall needed. Some representative annual amounts of rainfall needed for cropland in the western United States are 305 to 760 mm (12 to 30 inches) for cereal grains and 610 to 1,525 mm (24 to 60 inches) for forage. In the Near East,cotton needs about 915 mm (36 inches), whereas rice may require two to three times that amount. In humid regions of the United States, where irrigation supplements rainfall, grain crops may require 150 to 230 mm (6 to 9 inches) of water. In addition to satisfying the needs of the crop, allowances must be made for water lost directlyto evaporation and during transport to the fields.

Transport systems

The type of transport system used for an irrigation project is often determined by the source of the water supply. If a surface water supply is used, a large canal or pipeline system is usually required to carry the water to the farms because the reservoir is likely to be distant from the point of use. If subsurface water drawn from wells isused, a much smaller transport system is needed, though canals or pipelines may be used. The transport system will depend as far as possible on gravity flow, supplemented if necessary by pumping. From the mains, water flows into branches, or laterals, and finally to distributors that serve groups of farms. Many auxiliary structures are required, including weirs (flow-diversion dams), sluices, and other types of dams. Canals are normally linedwith concrete to prevent seepage losses, control weed growth, eliminate erosion hazards, and reduce maintenance. The most common type of concrete canal construction is by slip forming. In this type of construction, the canal is excavated to the exact cross section desired and the concrete placed on the earth sidesand bottom.

Pipelines may be constructed of many types of material. The larger lines are usually concrete whereas laterals may be concrete, cement–asbestos, rigid plastic, aluminum, or steel. Although

pipelines are more costly than open conduits, they do not require land after construction, suffer little evaporation loss, and are not troubled by algae growth.

Water application

After water reaches the farm it may be applied by surface, subsurface, or sprinkler-irrigation methods. Surface irrigation is normally used only where the land has been graded so that uniform slopes exist. Land grading is not necessary for other methods. Each method includes several variations, only the more common of which are considered here.

irrigation sprinklers

Irrigation sprinklers.

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...assed as either flood or furrow systems. In the flood ...ield and allowed to move over the entire surface to the ...ge crops are quite often irrigated by flood techniques. ...such as corn (maize), cotton, sugar beets, and potatoes. ...un in the furrows. In either type of surface irrigation ...r edge of the fields permit excess water to be removed

Sub irrigation is a less common method. An impermeable layer must be located below, but near, the root zone of the crop so that water is trapped in the root zone. If this condition exists, water is applied to the soil through tile drains or ditches.

Sprinklers have been used increasingly to irrigate agricultural land. Little or no preparation is needed, applicationrates can be controlled, and the system may be used for frost protection and the application of chemicals, such as pesticides, herbicides, and fertilizers. Sprinklers range from those that apply water in the form of a mist to those that apply an inch or more per hour.

Evaporation and seepage control

Various techniques have been tried to reduce losses of irrigation water. Two major sources of loss, particularly from surface supplies and surface systems, are evaporation and seepage from reservoirs and canals. Many studies have been made of techniques to suppress evaporation. One of the more promising appears to be application of a special alcohol film on the surface, which retards evaporation by about 30 percent and does not reduce the quality of the water. The primary problem in its use is that it is fragile; a strong wind can blow it apart and expose the water to evaporation.

Seepage has largely been controlled by lining main and distribution channels with impervious material, typicallyconcrete. Other materials used are asphalt and plastic film, though plastic tends to deteriorate if it is exposed to sunlight.

Typical systems

The typical surface irrigation system utilizes a publicly developed water supply—e.g., a river-basin reservoir. The public project also constructs the main canals to take water from the reservoir to the agricultural land. In general the canals flow by gravity, but lift stations are often required. Supply and field canals are used to bring the water to the individual field, where it is applied to the land either by furrow or by flooding method.

Until recently most sprinkler-irrigation systems depended on privately developed water supplies, but many modern sprinkler systems have been able to draw on public water supplies. In either case, a pump is required to pump water from a large well (3,785 litres, or 1,000 gallons, per minute or more) or a supply canal. The water goes into the system main and thence to a sprinkler unit. Many automatic or semiautomatic moving sprinkler systems travel over the field applying water. Two common units are the so-called centre pivot and the travelling sprinkler. The centre-pivot unit is anchored at the centre of the field; a long lateral (arm) with sprinklers mountedon it sweeps the field in a circle. The system has the disadvantage of missing the corners of a square field. A travelling sprinkler is mounted on a trailer and propelled across the field in a lane that has been left unplanted.

The unit drags a flexible hose connected to the main supply line. When it reaches the end of the lane, it is automatically shut off and can be moved to the next lane. Despite some shortcomings, all sprinkler systems are effective in applying a controlled amount of water at a high level of efficiency with a minimum of labour.

Modern drainage system planning and construction planning a system

The planning and design of drainage systems is not an exact science. Although there have been many advances insoil and crop science, techniques have not been developed for combining the basic principles involved into precise designs. One of the primary reasons for difficulty in applying known theory is the capricious variability ofnatural soil in contrast to the idealized soils required to develop a theory.

The type of drainage system designed depends on many factors, but the most important is the type of soil, which determines whether water will move through rapidly enough to use subsurface drainage. Soils that have a high percentage of sand- and silt-size particles and a low percentage of clay-size particles usually will transmit water rapidly enough to make subsurface drainage feasible. Soils that are high in clay-size particles usually cannot be drained by subsurface improvements. It is essential to consider soil properties to a depth of 1.5 to 1.8 metres (5 to6 feet) because the layer in the soil that transmits water the slowest controls the design, and subsurface improvements may be installed to these depths.

The topography or slope of the land is also important. In many cases, land in need of drainage is so flat that a contour map showing elevations 30 cm (12 inches) or 15 cm (6 inches) apart is used to identify trouble spots and possible outlets for drainage water. Often an outlet can be developed only by collective community action. The rainfall patterns, the crops to be grown, and the normal height of the water table also are considered. If heavy rainfall is not probable during critical stages of crop growth, less extensive drainage improvements may suffice. The capacity of the system is governed in part by the growth pattern of the crop, its planting date, critical stages of growth, tolerance of excess water, harvest date, and value.

In some areas the normal water level in the soil is high, in others low; this variable is always investigated before adrainage system is planned.

Types of drainage systems

Drainage systems may be divided into two categories, surface and subsurface. Each has several components with similar functions but different names. At the lower, or disposal, end of either system is an outlet. In order of decreasing size, the components of a surface system are the main collection ditch, field ditch, and field drain; and for a subsurface system, main, sub main, and lateral conduits from the sub main. The outlet is the point of disposalof water from the system; the main carries water to the outlet; the sub main or field ditch collects water from a number of smaller units and carries it to the main; and the lateral or field drain, the smallest unit of the system, removes the water from the soil.

The outlet for a drainage system may be a natural stream or river or a large constructed ditch. A constructed ditchusually is trapezoidal in section with side banks flat enough to be stable. Grass may be grown on the banks, which are kept clear of trees and brush that would interfere with the flow of water.

A surface drainage system removes water from the surface of the soil and to approximately the bottom of the field ditches. A surface system is the only means for drainage improvement on soils that transmit water slowly. Individual surface drains also are used to supplement subsurface systems by removing water from ponded areas.

The field drains of a surface system may be arranged in many patterns. Probably the two most widely used areparallel drains and random drains. Parallel drains are channels running parallel to one another at a uniform spacing of a few to several hundred meters apart, depending on the soil and the slope of the land. Random drainsare channels that run to any low areas in the field. The parallel system provides uniform drainage, whereas the random system drains only the low areas connected by channels. In either case the channels are shallow with flatsides and may be farmed like the rest of the field. Crops are usually planted perpendicular to the channels so that the water flows between the rows to the channels.

Some land grading of the fields where surface drains are installed is usually essential for satisfactory functioning.Land grading is the shaping of the field so that the land slopes toward the drainage channels. The slope may be uniform over the entire field or it may vary from part to part. Historically, the calculations for planning land grading were time-consuming, a factor that restricted the alternatives available for final design is necessary. Today, computer models rapidly explore many possibilities before a final land grading design is selected.

In a subsurface drainage system, often called a tile system, all parts except the outlet are located below the surface of the ground. It provides better drainage than a surface system because it removes water from the soil tothe depth of the drain, providing plants a greater mass of soil for root development, permitting the soil to warm up faster in the spring, and maintaining a better balance of bacterial action, the air in the soil, and other factors needed for maximum crop growth.

The smallest component of the subsurface system, the lateral, primarily removes water from the soil. The laterals may be arranged in either a uniform or a random pattern. The choice is governed by the crop grown and its value, the characteristics of the soil, and the precipitation pattern.

The primary decision required for a system with uniform laterals is their depth and spacing. In general, the deeper the laterals can be emplaced, the farther apart they can be spaced for an equivalent degree of drainage. Laterals usually are spaced from 24 to 91 metres (80 to 300 feet) apart and 0.9 to 1.5 metres (3 to 5 feet) deep.

Subsurface drainage systems are as important in many irrigated areas as they are in humid areas. A drainage system is needed on irrigated lands to control the water table and ensure that water will be able to move through a soil, thus keeping salts from accumulating in the root zone and making the soil unproductive.

Construction and maintenance

Most subsurface drains are constructed by excavating a trench, installing a tile, and backfilling the trench. Control of the machines to assure proper slope of the drain has been a major problem, but recent development in excavation technology, including the use of laser beams for grade control, have helped to solve it. Traditionally, clay or concrete tile has been the principal material used, but many types of perforated plastic tubes are now employed. An advantage is the reduction in weight of the material handled.

With proper maintenance, drainage systems give relatively long life. Selected herbicides are often applied to keep woody growth and water weeds out of the channels. Grates are usually installed over outlets to prevent rodents and burrowing animals from building nests.

Surface drainage systems need almost yearly maintenance to assure the slope and cross section of the channels and the slope of the graded areas because the slopes are so flat that small changes in the ground surface can makemarked changes in the ability of a system to function.

Subsurface systems need periodic inspection but usually require little servicing. The outlet of the system andinfrequent structural failure of the material are the usual points for service.

Land reclamation through irrigation and drainage

The continual need for increased food and fiber production requires the continued development of new agricultural lands and increased efficiency of existing agricultural areas. Development of new agricultural areas is rarely possible without irrigation or drainage systems or both. Easily recognized improvements are the large-scale river-basin projects designed for flood control, irrigation, and power generation. Such projects are in various stages of design or construction in many countries of the world—for example, China, India, Egypt, Iran, Australia, and the United States. In almost all cases drainage of the irrigated lands is considered a companion requirement. If possible, the drainage improvements are subsurface.

A combination of drainage and irrigation is used to reclaim large areas of land that have been abandoned because of salt accumulation. In this case subsurface drainage systems must be installed so that high water tables are lowered and pure water flushed through the soil, dissolving the salts and carrying them away in the drainage water. Large areas in the United States, India, and the Middle East are potentially available for reclamation bythis technique.

The people of the Netherlands have reclaimed land from the sea by the use of drainage. Since the IJsselmeer (formerly Zuiderzee) barrier dam was closed in 1932, converting this large body of water into a freshwater lake, the Dutch have been continually enclosing and reclaiming smaller bodies (polders). After dikes are built around a polder, the area is drained by pumping out the water. Drainage channels and, in many places, subsurface drains are installed so that the root zone of crops can be drained. After this, cropping is started as the last step in the reclamation process.

The development of land-clearing machinery and surface-drainage techniques has made it possible to clear and drain tropical lands for agricultural production. The first step is the removal of trees, brush, and other tropical growth. Outlet ditches are constructed, followed by drains. In some cases subsurface drains are possible, but more often the soils and rainfall conditions combine to make this improvement impractical. Surface drains are installedon a uniform pattern and the land is smoothed or graded. Drainage systems on newly reclaimed tropical land require special attention while the soils are stabilizing, and some reconstruction is often needed after the soil stabilization is complete.

Irrigation and drainage throughout the world

The Food and Agriculture Organization of the United Nations (FAO) keeps the most complete statistics on irrigated lands; it estimates that in the entire world some 275 million hectares (680 million acres) are irrigated. More than 130 countries report some acreage under irrigation. Asia irrigates close to 70 percent of the total area of the world that is irrigated; most of this is the large surface-irrigated rice-producing areas of China, India, Pakistan, and Southeast Asia. Sub-Saharan Africa has the lowest percentage of its cultivated lands irrigated. Sprinkler and localized irrigation methods are employed throughout the world and account for about 14 percentof the total area of irrigated land.

Statistics on drainage improvements are sparser than statistics on irrigation. It may safely be said that drainage in one form or another is practiced in almost every country of the world. It is now universally accepted that drainage is needed as much on irrigated as on non irrigated land. Countries such as India that have large-scale river-basin developments planned with irrigation also have companion drainage systems planned so that the land will not be damaged by salt accumulation. It is almost certain that the land area of the world improved by irrigation and drainage will continue to increasebecause these practices are two of the most elemental means of reclaiming and improving agricultural lands.

3.2 Ground Water System

GROUND WATER RESOURCES

The amount of water stored in the earth's crust may be of the order of 8 billion cubic kilometres, half of which is at depths less than 800 m. This water inside the earth is about 35 times the combined storage of all the world's rivers, fresh water lakes, reservoirs, and inland seas, and is about one-third the volume of water stored in the arctic and antarctic ice fields, the glaciers of Greenland, and the great mountain systems of the world. All of this ground water, however, cannot be utilised because of physiographic limitations.

The estimate of the present ground water resource in India (3) is of the order of 650 cubic kilometres (as against 1880 cubic km for surface water resources), out of which utilisable ground water is assessed at around 420 cubic km (as against 690 cubic km for surface water resources); see Table 1.4. Ground water is that part of the subsurface water which occurs within the saturated zone of the earth's crust where all pores are filled with water. Ground water has also been referred to as that part of the subsurface water which can be lifted or which flows naturally to the earth's surface. A hole or shaft, usually vertical, is excavated in the earth to lift ground water to the earth's surface and is termed a well. A well can also be used for disposal of water, artificial recharge, draining out agricultural lands, and relieving pressures under hydraulic structures. The Chinese are known to be the first to have drilled deep wells using bamboo rods tipped with iron . The rods were lifted and dropped manually and the method was similar to the method now known as cable tool drilling. Ground water flows to the earth's surface through naturally discharging springs and streams and rivers which are sustained by ground water itself when overland runoff is not present. Following significant features of ground water should always be kept in mind while managing ground water:

Ground water is a huge water resource, but is exhaustible and is unevenly available.

Ground water and surface water resources are interrelated and, hence, should be considered together.

Excessive and continued exploitation of ground water must be avoided as natural replenishment of the ground water resource is a very slow process.

Ground water is generally better than surface water in respect of biological characteristics. On the other hand, surface water is generally better than ground water in terms of chemical characteristics.

Ground water may be developed in stages on ''pay-as-you-go'' or ''pay-as-you-grow'' basis. Surface water development usually needs large initial capital investment.

Underground reservoirs storing ground water are more advantageous than surface reservoirs.

There is no construction cost involved in underground reservoirs. But, well construc- tion, pumps and energy for pumping water, and maintenance of pumps and wells require money.

Underground reservoirs do not silt up, but surface siltation of recharge areas may appreciably reduce recharge rates.

The evaporation from underground reservoirs is much less.

Underground reservoirs do not occupy the land surface which may be useful for some other purposes.

Ground water is generally of uniform temperature and mineral quality and is free of suspended impurities.

Ground water source has indefinite life, if properly managed.

Ground water source is replenished through the processes of infiltration and percolation. Infiltration is the process by which the precipitation and surface water move downward into the soil. Percolation is the vertical and lateral movement through the various openings in the geological formations. Natural sources of replenishment include rainwater, melting snow or ice and water in stream channels, and lakes or other natural bodies of water. Rainwater may infiltrate into the ground directly or while flowing over the land enroute to a river, or stream, or other water bodies. Artificial sources of replenishment (or recharge) include the following (2):

Leakage from reservoirs, conduits, septic tanks, and similar water related struc-tures.

Irrigation, or other water applications including deliberate flooding of a naturally porous area.

Effluents discharged to evaporation or percolation ponds.

Injection through wells or other similar structures.

WELL IRRIGATION

In view of the large amount of utilisable ground water, higher agricultural yield of tubewell- irrigated lands in comparison to that of canal-irrigated lands (see Table 1.3), and favourable impact of its use on waterlogging, it is only logical to develop ground water resources for irrigation and other activities. Most of the existing canal systems in India are of a protective nature, i.e., they provide protection against famine. They were not designed to promote intensive farming. Well irrigation ensures more reliable irrigation and, therefore, enables the farmers to grow more remunerative crops with improved yield. The following are the main requirements for the success of well irrigation:

Presence of a suitable aquifer which can yield good quality water in sufficient quan-tity.

Availability of energy, preferably electric power, for pumps.

Well distributed demand for irrigation throughout the year.

Suitable configuration of command area with the highest ground around the centre of the command area.

In general, well irrigation is more efficient than canal irrigation. The following are the comparative features of the two types of irrigation:

In the canal irrigation system, major structures, such as headworks, main and branch canals, etc. must be constructed prior to the start of proportionate agricultural activ- ity which grows gradually because of the availability of irrigation facility. But, wells can be constructed gradually to keep pace with the development of the agricultural activities of the area.

Transit losses in well irrigation are much less than those in canal irrigation system.

Isolated patches of high lands can be better served by well irrigation.

Well irrigation offers an effective anti-waterlogging measure of the affected lands and reduces the chances of waterlogging of canal-irrigated lands.

Well irrigation ensures relatively more reliable supply of water at the time of need. This results in better yield. Besides, farmers can switch over to more remunerative crops due to the availability of assured supply.

Well irrigation needs energy for pumping. Installation and maintenance of pumps and the cost of running the pumps make well irrigation costlier.

Failure of power supply at the time of keenest demand may adversely affect the yield in case of well irrigation systems.

It is thus obvious that both irrigation systems have advantages as well as disadvantages. Therefore, both must be used in a judicious manner to obtain maximum benefits, such that there is no waterlogging and the ground water resource can be maintained indefinitely.

OCCURRENCE OF GROUND WATER

The subsurface medium within which ground water occurs is either porous or fractured or both. The subsurface occurrence of ground water can be divided into two zones (Fig. 4.1): (i) the vadose zone or unsaturated zone or zone of aeration, and (ii) the phreatic zone or saturated zone or zone of saturation. In the saturated zone, all pores or voids are filled with water whereasin the unsaturated zone, pores contain gases (mainly air and water vapours) in addition to water.

Soil water

Unsaturated zoneor
Zone of aerationor
Vadose zone

Intermediate zone

Water table

Capillary water

Zone of saturationor
Phreatic zone
or Saturated zone

Phreatic water (Ground water)

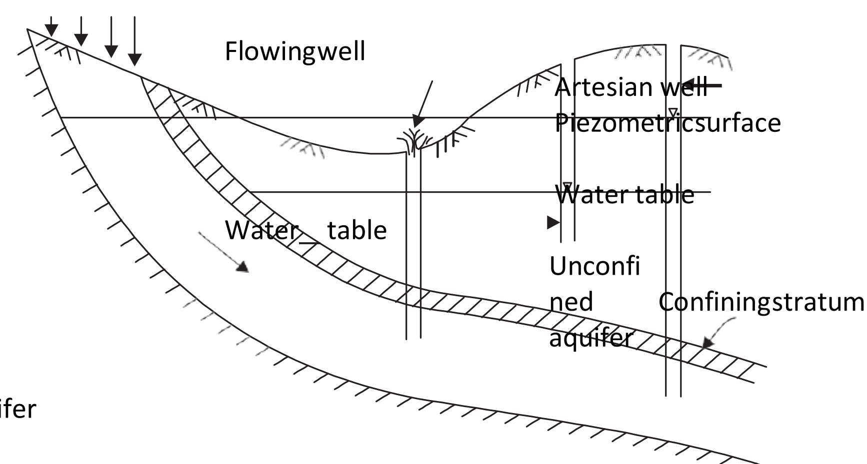
Fig. 4.1 Vertical distribution of subsurface water

The water table is defined as the upper limit of the saturated zone. However, it should be noted that all the pores near the base of the capillary water zone (which itself may range from practically nothing in coarse material to about 2.5 m or more in clay materials) may be completely saturated. The number of pores filled with water decreases in the upward direction of the capillary water zone. One can, therefore, expect the upper limit of actual saturation to be an irregular surface. Water table should, therefore, be redefined as the upper limit of saturation at atmospheric pressure.

The saturated zone containing interconnected pores may exceed depths penetrated by oil wells (more than 12,000 m). However, freshwater (part of the hydrologic cycle) is found only up to depths of about 800 m (2).

A saturated geologic formation capable of yielding water economically in sufficient quantity is known as an aquifer (or water-bearing formation or ground water reservoir). Ground water constantly moves through an aquifer under local hydraulic gradients. Thus, aquifers perform storage as well as conduit functions. Ground water may exist in aquifers in two different manners: (i) unconfined, and (ii) confined. The unconfined condition occurs when the water table is under atmospheric pressure and is free to rise or fall with changes in the volume of the stored water. An aquifer with unconfined conditions is referred to as an unconfined or water table aquifer. An aquifer which is separated from the unsaturated zone by an impermeable or very less permeable formation is known as confined aquifer (or artesian aquifer or pressure aquifer). Ground water in a confined aquifer is under pressure which is greater than the atmospheric pressure. The water level in a well penetrating a confined aquifer indicates the piezometric pressure at that point and will be above the bottom of the upper confining formation. Such wells are known as artesian wells and if the water level rises above the land surface, a flowing well results (Fig. 4.2).

Recharge area



well

Confinedaquifer

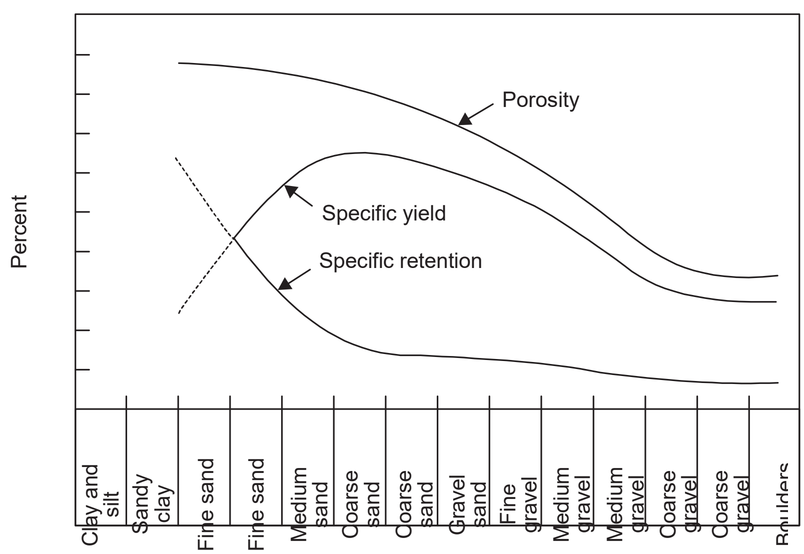
Impermeablestratum

Fig. 4.2 Aquifers and wells

Water released from an unconfined aquifer is the result of dewatering or draining of the aquifer material. In the case of confined aquifer, the release of water is the result of a slight expansion of water and a very small compression of the porous medium.

The availability, movement, and quality of ground water depend mainly on the characteristics of the medium. The following characteristics of the medium affect the availabilityand movement of ground water.
Porosity can be defined as the ratio of the volume of pores to the total volume of the porous medium. It ranges from 0 to 50 per cent for most of the rock materials. For aquifer considerations, porosities less than 5% are considered small, those between 5% and 20% are considered medium and those greater than 20% are considered large (2). Porosity is, obviously, an inherent characteristic of the material independent of the presence or absence of water. For ground water studies, the interconnected pore space which can be drained by gravity should be used for determining the porosity and such porosity is known as effective porosity.
The specific yield of a soil formation is defined as the ratio of the volume of water which the soil formation, after being saturated, will yield by gravity to the volume of the soil formation.
The specific retention of a soil formation is defined as the ratio of the volume of water which the soil formation, after being saturated, will retain against the pull of gravity to the volume of the soil formation.
These definitions of specific yield and specific retention implicitly assume complete drainage. Obviously, the sum of the specific yield and the specific retention would be equal to the porosity of the given soil formation. The product of the average specific yield of a saturated water-bearing formation and its total volume gives the volume of water which can be recovered from the formation by gravity drainage. It may be noted that the time factor is not included in the definition of specific yield. However, the gravity draining of a formation decreases with time and may continue for years. Fine-grained materials may have lesser specific yield than coarse materials even though their porosity may be greater (Fig. 4.3 and Table 4.1).

50
45
40
35
30
25
20
15
10
5
0



1/16 1/8 1/4 1/2 1 2 4 8 16 32 64 128 256
Grain size, d_{90} millimetres

Fig. 4.3 Typical variation of porosity, specific yield, and specific retention with grain size (4)

Table 4.1 Representative porosity and specific yield of selected earth material

Material	Porosity %	Specific yield %
Clay	45 – 55	1 – 10
Sand	25 – 40	10 – 30
Gravel	25 – 40	15 – 30
Sand and gravel	10 – 35	15 – 25
Sandstone	5 – 30	5 – 15
Shale	0 – 10	0.5 – 5
Limestone	1 – 20	0.5 – 5

In case of confined aquifers there is no dewatering or draining of the material unless the hydraulic head drops below the top of the aquifer. Therefore, the concept of specific yield does not apply to confined aquifers and an alternative term, storage coefficient or storativity is used for confined aquifers. Storativity or storage coefficient is defined as the volume of water an aquifer would release from or take into storage per unit surface area of the aquifer for a unit change in head. Its value is of the order 5×10^{-2} to 1×10^{-5} (2). For the same drop in head, the yield from an unconfined aquifer is much greater than that from a confined aquifer.

The permeability of a porous medium describes the ease with which a fluid will pass through it. Therefore, it depends on the characteristics of the medium as well as the flowing fluid. It would be logical to use another term which reflects only the medium characteristics. This term is named intrinsic permeability, and is independent of the properties of the flowing fluid and depends only on the characteristics of the medium. It is proportional to the square of the representative grain diameter of the medium, and the constant of proportionality depends on porosity, packing, size distribution, and shape of grains.

The permeability of a medium is measured in terms of hydraulic conductivity (also known as the coefficient of permeability) which is equal to the volume of water which flows in unit time through a unit cross-sectional area of the medium under a unit hydraulic gradient at the prevailing temperature. The hydraulic conductivity, therefore, has the dimensions of [L/T] and is usually expressed as metres per day or metres per hour. It should be noted that an unsaturated medium would have lower hydraulic conductivity because of the resistance to a flow of water offered by the air present in the void spaces.

The transmissivity, a term generally used for confined aquifers, is obtained by multiplying the hydraulic conductivity of an aquifer with the thickness of the saturated portion of the aquifer. It represents the amount of water which would flow through a unit width of the saturated portion of the aquifer under a unit hydraulic gradient and at the prevailing temperature.

FLOW OF WATER THROUGH POROUS MEDIA

Ground water flows whenever there exists a difference in head between two points. This flow can either be laminar or turbulent. Most often, ground water flows with such a small velocity that the resulting flow is laminar. Turbulent flow occurs when large volumes of water converge through constricted openings as in the vicinity of wells.

Based on a series of experiments conducted in the vertical pipe filled with sand, Henry Darcy, a French engineer, in 1856 concluded that the rate of flow, Q through a column of saturated sand is proportional to the difference in hydraulic head, Δh, between the ends of the column and to the area of flow cross-section A, and inversely proportional to the length of the column, L. Thus,

$$Q = KA\frac{\partial h}{L} \qquad (4.1)$$

Here, K is the constant of proportionality and is equal to the hydraulic conductivity of the medium. Equation (4.1) is known as Darcy's law and can also be written as

$$V = K\frac{\partial h}{L} \qquad (4.2)$$

in which, V is the specific discharge (or the apparent velocity of flow) and $\frac{\partial h}{L}$ is the hydraulic gradient. Expressed in general terms, Darcy's law, Eq. (4.2), becomes

$$V = -K\frac{dh}{ds} \qquad (4.3)$$

in which, dh/ds is the hydraulic gradient which is negative, since h decreases in the positive direction of the flow. Thus, flow along the three principal co-ordinate axes can be described as

$$u = -K_x\frac{\partial h}{\partial x} \qquad (4.4a)$$

$$v = -K_y\frac{\partial h}{\partial y} \qquad (4.4b)$$

and $$w = -K_z\frac{\partial h}{\partial z} \qquad (4.4c)$$

Here, u, v, and w are the velocity components in the x-, y-, and z-directions, respectively, and K_x, K_y, and K_z are hydraulic conductivities (coefficients of permeability) in these directions.

In Darcy's law, the velocity is proportional to the first power of the hydraulic gradient and is, therefore, applicable to laminar flows only. For a flow through porous medium, Reynolds number R_e can be expressed as

$$R_e = \frac{Vd\rho}{\mu}$$

Here, d is the representative average grain diameter which approximately represents the average pore diameter, i.e., the flow dimension. ρ and μ are, respectively, the mass density and the dynamic viscosity of the flowing water. An upper limit of Reynolds number ranging between 1 and 10 has been suggested as the limit of validity of Darcy's law (4). A range rather than a unique value of R_e has been specified in view of the possible variety of grain shapes, grain-size distribution, and their packing conditions. For natural ground water motion, R_e is usually less than unity and Darcy's law is, therefore, usually applicable.

When Darcy's law is substituted in the continuity equation of motion, one obtains the equation governing the flow of water through a porous medium. The resulting equations for confined and unconfined aquifers are, respectively, as follows (5):

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2} = \frac{S}{T}\frac{\partial h}{\partial t} \qquad (4.5)$$

and

$$\frac{\partial^2 H^2}{\partial x^2} + \frac{\partial^2 H^2}{\partial y^2} = \frac{2n}{K}\frac{\partial H}{\partial t} \qquad (4.6)$$

Here, H represents the hydraulic head in unconfined aquifer and n is the porosity of the medium. Equations (4.5) and (4.6) are, respectively, known as Boussinesq's and Dupuit's equations. Both these equations assume that the medium is homogeneous, isotropic, and water is incompressible. Equation (4.5) also assumes that large pressure variations do not occur. Equation (4.6) further assumes that the curvature of the free surface is sufficiently small for the vertical components of the flow velocity to be negligible in comparison to the horizontal component. For steady flow, Eqs. (4.5) and (4.6) become

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2} = 0 \qquad (4.7a)$$

$$\frac{\partial^2 H^2}{\partial x^2} + \frac{\partial^2 H^2}{\partial y^2} = 0 \qquad (4.7b)$$

A well is a hydraulic structure which, if properly designed and constructed, permits economic withdrawal of water from an aquifer (6). When water is pumped from a well, the water table (or the piezometric surface in case of a confined aquifer) is lowered around the well. The surface of a lowered water table resembles a cone and is, therefore, called the cone of depression. The horizontal distance from the centre of a well to the practical limit of the cone of depression is known as the radius of influence of the well. It is larger for wells in confined aquifers than for those in unconfined aquifers. All other variables remaining the same, the radius of influence is larger in aquifers with higher transmissivity than in those with lower transmissivity. The difference, measured in the vertical direction, between the initial water table (or the piezometric surface in the confined aquifer) and its lowered level due to pumping at any location within the radius of influence is called the drawdown at that location. Well yield is defined as the volume of water discharge, either by pumping or by free flow, per unit time. Well yield per unit drawdown in the well is known as the specific capacity of the well.

With the continued pumping of a well, the cone of depression continues to expand in an extensive aquifer until the pumping rate is balanced by the recharge rate. When pumping and recharging rates balance each other, a steady or equilibrium condition exists and there is no further drawdown with continued pumping. In some wells, the equilibrium condition may be attained within a few hours of pumping, while in others it may not occur even after prolonged pumping.

Equilibrium Equations

For confined aquifers, the governing equation of flow, Eq. (4.7a), can be written in polar cylindrical coordinates (r, θ, z) as

$$\frac{1}{r}\frac{\partial}{\partial r}\left(r\frac{\partial h}{\partial r}\right) + \frac{1}{r^2}\frac{\partial^2 h}{\partial \theta^2} + \frac{\partial^2 h}{\partial z^2} = 0 \qquad (4.8)$$

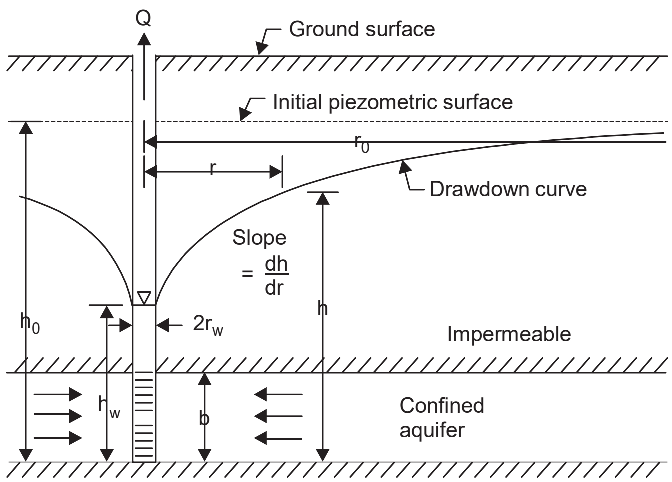
If one assumes radial symmetry (i.e., h is independent of θ) and the aquifer to be horizontal and of constant thickness (i.e., h is independent of z), Eq. (4.8) reduces to

$$\frac{d}{dr}\left(r\frac{dh}{dr}\right) = 0 \qquad (4.9)$$

For flow towards a well, penetrating the entire thickness of a horizontal confined aquifer, Eq. (4.9) needs to be solved for the following boundary conditions (Fig. 4.4):

at $r = r_0$, $h = h_0$ (r_0 is the radius of influence)

at $r = r_w$, $h = h_w$ (r_w is the radius of well)



Impermeable

Fig. 4.4 Radial flow to a well penetrating an extensive confined aquifer

On integrating Eq. (4.9) twice with respect to r, one obtains

$$r \frac{dh}{dr} = C_1$$

and $\quad h = C_1 \ln r + C_2 \qquad (4.10)$

in which C_1 and C_2 are constants of intergration to be obtained by substituting the boundaryconditions in Eq. (4.10) which yields

$h_0 = C_1 \ln r_0 + C_2$

and $\quad h_w = C_1 \ln r_w + C_2$

Hence, $C_1 = \frac{h_0 - h_w}{\ln (r_0/r_w)}$

and $C_2 = h_0 - \frac{h_0 - h_w}{\ln (r_0/r_w)} \ln r_0$

Also, $C_2 = h_w - \frac{h_0 - h_w}{\ln (r_0/r_w)} \ln r_w$

Finally, $h = h_0 - \frac{h_0 - h_w}{\ln (r_0/r_w)} \ln (r_0/r)$ (4.11)

and also, $h = h_w + \frac{h_0 - h_w}{\ln (r_0/r_w)} \ln (r/r_w)$ (4.12)

Further, the discharge Q through any cylinder of radius r and height equal to the thickness of the aquifer B is expressed as

$$Q = -K(2\pi r B)\frac{dh}{dr}$$

$$= -2\pi T\left(r\frac{dh}{dr}\right)$$

$$= -2\pi T C_1$$

$$Q = -2\pi T \frac{h_0 - h_w}{\ln (r_0/r_w)} \quad (4.13)$$

Thus, Eqs. (4.11) and (4.12) can be rewritten as

$$h = h_0 + \frac{Q}{2\pi T} \ln (r_0/r) \quad (4.14)$$

and $$h = h_w - \frac{Q}{2\pi T} \ln (r/r_w) \quad (4.15)$$

It should be noted that the coordinate r is measured positive away from the well and that the discharge towards the well is in the negative direction of r. Therefore, for a discharging well, Q is substituted as a negative quantity in Eqs. (4.13) through (4.15). If the drawdown atany radial distance r from the well is represented by s, then

$$s = h_0 - h = -\frac{Q}{2\pi T} \ln r_0/r \quad (4.16)$$

and the well drawdown s_w is given as

$$s_w = h_0 - h_w = -\,Q \quad \ln r_0/r_w$$

(4.17)

w 0 w 2 T 0 w

For unconfined aquifers, one can similarly obtain the following equations starting from the Dupuit's equation, Eq. (4.7b):

$$H^2 = H_0^2 - \frac{H_0^2 - H_w^2}{\ln (r_0/r_w)} \ln (r_0/r) \quad (4.18)$$

$$H^2 = H_w^2 + \frac{H_0^2 - H_w^2}{\ln (r_0/r_w)} \ln (r/r_w) \quad (4.19)$$

$$Q = -\pi K \frac{H_0^2 - H_w^2}{\ln (r_0/r_w)} \quad (4.20)$$

$$H^2 = H_0^2 + \frac{Q}{\pi K} \ln (r_0/r) \quad (4.21)$$

$$H^2 = H_w^2 - \frac{Q}{\pi K} \ln (r/r_w) \quad (4.22)$$

Example 4.3 A well with a radius of 0.3 m, including gravel envelope and developed zone, completely penetrates an unconfined aquifer with K = 25 m/day and initial water table at 30 m above the bottom of the aquifer. The well is pumped so that the water level in the well remains at 22 m above the bottom of the aquifer. Assuming that pumping has essentially no effect on water table height at 300 m from the well, determine the steady-state well discharge.Neglect well losses.

Solution: From Eq. (4.20),

$$Q = -\pi K \frac{H_0^2 - H_w^2}{\ln (r_0/r_w)}$$

$$= -\frac{3.14 \times 25 \times (30^2 - 22^2)}{\ln \left(\frac{300}{0.3}\right)}$$

= – 4729.84 m^3/day.

Negative sign indicates pumping well.

Non-Equilibrium Equations

For an unsteady flow in confined aquifer, Eq. (4.5) can be written in polar cylindrical coordinates(r, θ, z) as

$$\frac{1}{r}\frac{\partial}{\partial r}\left(r \frac{\partial h}{\partial r}\right) + \frac{1}{r^2}\frac{\partial^2 h}{\partial \theta^2} + \frac{\partial^2 h}{\partial z^2} = \frac{S}{T}\frac{\partial h}{\partial t} \quad (4.23)$$

which reduces to

$$\frac{\partial^2 h}{\partial r^2} + \frac{1}{r}\frac{\partial h}{\partial r} = \frac{S}{T}\frac{\partial h}{\partial t} \quad (4.24)$$

if one assumes radial symmetry and the aquifer to be horizontal and of constant thickness.

The general form of solution of Eq. (4.24) is h(r, t). For unsteady flow towards a well penetrating the entire thickness of a confined aquifer, Eq. (4.24) needs to be solved for the following boundaryconditions:

(i) $h(\infty, t) = h_0$

$$2\pi r B \left(K \frac{\partial h}{\partial r}\right) = -Q \text{ for } t > 0 \text{ (flux condition)}$$

$$\lim_{r_w \to 0}\left(r_w \frac{\partial h}{\partial r} \right) = -\frac{Q_w}{2\pi T_w}$$

which can be approximated as

$$\lim_{r \to 0}\left(r \frac{\partial h}{\partial r}\right) = -\frac{Q}{2\pi T}$$

$h(r, 0) = h_0$ (initial condition).

Theis (7) obtained a solution of Eq. (4.24) by assuming that the well is replaced by a mathematical sink of constant strength. The solution is expressed as

$$s = h_0 - h = -\frac{Q}{4\pi T}\int_u^{\infty} \frac{e^{-u}}{u}\, du \qquad (4.25)$$

in which $u = \dfrac{r^2 S}{4Tt}$

where, t is the time since the beginning of pumping. Equation (4.25) is also written as

$$s = -\frac{Q}{4\pi T} W(u) \qquad (4.26)$$

in which, W(u) is known as the well function (Table 4.2) and is expressed as a function of u inthe form of the following convergent series:

$$W(u) = -0.5772 - \ln u + u - \frac{u^2}{2 \cdot 2!} + \frac{u^3}{3 \cdot 3!} - \frac{u^4}{4 \cdot 4!} + \ldots \qquad (4.27)$$

An approximate form of the Theis equation (i.e., Eq. (4.25)) was obtained by Cooper and Jacob (8) dropping the third and higher order terms of the series of Eq. (4.27). Thus,

$$s = -\frac{Q}{4\pi T}[-0.5772 - \ln u]$$

or $$s = -\frac{Q}{4\pi T}\left(\ln \frac{0.25\, Tt}{r^2 S}\right)$$

$$\therefore \quad s = -\frac{0.183\, Q}{T}\left(\log \frac{2.25\, Tt}{r^2 S}\right) \qquad (4.28)$$

For values of u less than 0.05, Eq. (4.28) gives practically the same results as obtained by Eq. (4.26). Note that Q is to be substituted as a negative quantity for a pumping well.

Because of the non-linear form of Eq. (4.6), its solution is difficult. Boulton (9) has presented a solution for fully penetrating wells in an unconfined aquifer. The solution is validif the water depth in the well exceeds 0.5 H_0. The solution is

$$s = \frac{Q}{2\pi K H_0}(1 + C_k)\, V(t', r') \qquad (4.29)$$

in which, C_k is a correction factor which can be taken as zero for t' less than 5, and according to Table 4.3 for t' greater than 5 (when C_k depends only on r'). V (t', r') is Boulton's well function dependent on r' and t' defined as

$$t' = \frac{Kt}{SH_0}$$

and $$r' = \frac{r}{H_0}$$

Table 4.2 Well function W(u) for different values of u

u	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0
× 1	0.219	0.049	0.013	0.0038	0.0011	0.00036	0.00012	0.000038	0.000012
$\times 10^{-1}$	1.82	1.22	0.91	0.70	0.56	0.45	0.37	0.31	0.26
$\times 10^{-2}$	4.04	3.35	2.96	2.68	2.47	2.30	2.15	2.03	1.92
$\times 10^{-3}$	6.33	5.64	5.23	4.95	4.73	4.54	4.39	4.26	4.14
$\times 10^{-4}$	8.63	7.94	7.53	7.25	7.02	6.84	6.69	6.55	6.44
$\times 10^{-5}$	10.94	10.24	9.84	9.55	9.33	9.14	8.99	8.86	8.74
$\times 10^{-6}$	13.24	12.55	12.14	11.85	11.63	11.45	11.29	11.16	11.04
$\times 10^{-7}$	15.54	14.85	14.44	14.15	13.93	13.75	13.60	13.46	13.34
$\times 10^{-8}$	17.84	17.15	16.74	16.46	16.23	16.05	15.90	15.76	15.65
$\times 10^{-9}$	20.15	19.45	19.05	18.76	18.54	18.35	18.20	18.07	17.95
$\times 10^{-10}$	22.45	21.76	21.35	21.06	20.84	20.66	20.50	20.37	20.25
$\times 10^{-11}$	24.75	24.06	23.65	23.36	23.14	22.96	22.81	22.67	22.55
$\times 10^{-12}$	27.05	26.36	25.96	25.67	25.44	25.26	25.11	24.97	24.86
$\times 10^{-13}$	29.36	28.66	28.26	27.97	27.75	27.56	27.41	27.28	27.16
$\times 10^{-14}$	31.66	30.97	30.56	30.27	30.05	29.87	29.71	29.58	29.46
$\times 10^{-15}$	33.96	33.27	32.86	32.58	32.35	32.17	32.02	31.88	31.76

Table 4.3 Values of C_k for t⍰ > 5

r⍰	0.03	0.04	0.06	0.08	0.1	0.2	0.4	0.6	0.8	1	2	4
C	– 0.27	– 0.24	– 0.19	– 0.16	– 0.13	– 0.05	– 0.02	0.05	0.05	0.05	0.03	0

Values of the function V have been tabulated in Table 4.4. The approximate values of V can also be calculated as follows: For $t_D < 0.01$ and $t_D/r_D > 10$.

$$V \approx \ln (2t_D/r_D)$$

For $t_D < 0.01$,

$$V \approx \sin h^{-1} (t_D/r_D) - 1$$

For $t_D < 0.05$,

$$V \approx \sin h^{-1} \left(\frac{1}{r_D}\right) + \sin h^{-1} \frac{[t_D/r_D]}{\sqrt{1 + r_D^2}} - \sin h^{-1} \left(\frac{1 - t_D}{r_D}\right)$$

For $t_D > 5.0$,

$$V \approx \frac{1}{2} W(u)$$

in which, $u = \dfrac{r^2 n_e}{4Tt}$

Here, n_e is the effective porosity of the aquifer.

The well equations mentioned in this section are valid only for a single well of small diameter having no storage capability and fully penetrating an extensive aquifer. The equations will be modified for the effect of partial penetration, well storage, bounded aquifers, interferenceof adjacent wells, and multi-layer aquifer systems.

Example 4.4 A fully penetrating artesian well is pumped at a rate Q = 1500 m^3/day from an aquifer whose storage coefficient and transmissivity are 4×10^{-4} and 0.145 m^2/min, respectively. Find the drawdowns at a distance 3 m from the production well after one hour ofpumping and at a distance of 350 m after one day of pumping.

Solution:

At r = 3 m and t = 1 h,

$$u = \frac{r^2 S}{4Tt} = \frac{3 \times 3 \times 4 \times 10^{-4}}{4 \times 0.145 \times (1 \times 60)} = 1.03 \times 10^{-4}$$

$\therefore$ w (u) = 8.62

From Eq. (4.26)

$$\therefore \quad s = \frac{Q}{4\pi T} W(u) = \frac{1500 / (24 \times 60)}{4 \times 3.14 \times 0.145} \times 8.62$$

= 4.93 m

Similarly, at r = 350 m and t = 1 day

Table 4.4 Function V(t′, r′) for different values of t′ and r′

t'	Value of r'																	
	0.001	*0.002*	*0.003*	*0.004*	*0.005*	*0.006*	*0.007*	*0.008*	*0.009*	*0.01*	*0.02*	*0.03*	*0.04*	*0.05*	*0.06*	*0.07*	*0.08*	*0.09*
0.01	2.99	2.30	1.90	1.64	1.42	1.28	1.15	1.04	0.95	0.875	0.474	0.322	0.240	0.192	0.158	0.135	0.118	0.104
0.02	3.68	2.97	2.58	2.30	2.09	1.92	1.76	1.64	1.52	1.42	0.860	0.610	0.468	0.378	0.316	0.270	0.236	0.210
0.03	4.08	3.40	3.00	2.70	2.46	2.28	2.13	2.00	1.88	1.79	1.18	0.860	0.675	0.555	0.465	0.400	0.350	0.310
0.04	4.35	3.68	3.26	2.98	2.75	2.58	2.42	2.29	2.17	2.06	1.42	1.07	0.850	0.710	0.600	0.525	0.460	0.410
0.05	4.58	3.90	3.49	3.20	2.96	2.79	2.64	2.50	2.38	2.28	1.60	1.24	1.010	0.850	0.725	0.630	0.560	0.500
0.06	4.76	4.06	3.65	3.36	3.15	2.96	2.80	2.68	2.56	2.45	1.78	1.40	1.15	0.970	0.840	0.735	0.650	0.585
0.07	4.92	4.20	3.80	3.51	3.30	3.12	2.96	2.82	2.70	2.60	1.91	1.54	1.28	1.09	0.950	0.835	0.740	0.670
0.08	5.08	4.34	3.94	3.65	3.42	3.24	3.09	2.95	2.84	2.72	2.04	1.65	1.39	1.20	1.04	0.925	0.825	0.750
0.09	5.18	4.47	4.05	3.75	3.54	3.35	3.20	3.05	2.95	2.84	2.14	1.75	1.50	1.29	1.14	1.02	0.910	0.825
0.1	5.24	4.54	4.14	3.85	3.63	3.45	3.30	3.15	3.04	2.94	2.25	1.85	1.58	1.38	1.22	1.09	0.985	0.890
0.2	5.85	5.15	4.78	4.50	4.28	4.10	3.93	3.80	3.66	3.56	2.87	2.46	2.20	1.98	1.80	1.65	1.52	1.42
0.3	6.24	5.50	5.12	4.85	4.61	4.43	4.28	4.14	4.01	3.90	3.24	2.84	2.54	2.32	2.14	1.98	1.85	1.74
0.4	6.45	5.75	5.35	5.08	4.85	4.67	4.50	4.38	4.26	4.15	3.46	3.05	2.76	2.54	2.36	2.20	2.07	1.96
0.5	6.65	6.00	5.58	5.25	5.00	4.85	4.70	4.55	4.45	4.30	3.65	3.24	2.95	2.72	2.52	2.38	2.24	2.14
0.6	6.75	6.10	5.65	5.40	5.15	4.98	4.82	4.68	4.56	4.45	3.76	3.37	3.09	2.85	2.67	2.50	2.38	2.26
0.7	6.88	6.20	5.80	5.50	5.25	5.08	4.92	4.80	4.68	4.55	3.90	3.50	3.20	2.99	2.80	2.64	2.50	2.38
0.8	7.00	6.25	5.85	5.60	5.35	5.20	5.00	4.90	4.80	4.65	3.96	3.55	3.26	3.05	2.86	2.71	2.58	2.46
0.9	7.10	6.35	6.00	5.70	5.50	5.30	5.12	5.00	4.90	4.75	4.05	3.65	3.36	3.15	2.96	2.80	2.66	2.55
1	7.14	6.45	6.05	5.75	5.55	5.35	5.20	5.05	4.95	4.83	4.10	3.74	3.45	3.22	3.04	2.90	2.75	2.64
2	7.60	6.88	6.45	6.15	5.92	5.75	5.60	5.50	5.35	5.25	4.59	4.18	3.90	3.68	3.50	3.34	3.20	3.09
3	7.85	7.15	6.70	6.45	6.20	6.00	5.85	5.75	5.60	5.50	4.82	4.42	4.12	3.90	3.72	3.57	3.45	3.31
4	8.00	7.28	6.85	6.58	6.35	6.15	6.00	5.90	5.75	5.70	4.95	4.55	4.26	4.04	3.86	3.70	3.59	3.46
5	8.15	7.35	7.00	6.65	6.50	6.25	6.10	6.00	5.85	5.80	5.05	4.68	4.40	4.19	4.00	3.85	3.71	3.60
6	8.20	7.50	7.10	6.75	6.55	6.35	6.20	6.10	5.95	5.85	5.20	4.78	4.50	4.26	4.09	3.92	3.80	3.69
7	8.25	7.55	7.15	6.85	6.62	6.40	6.30	6.20	6.05	5.95	5.25	4.85	4.58	4.35	4.18	4.00	3.90	3.78
8	8.30	7.60	7.20	6.90	6.70	6.50	6.35	6.25	6.10	6.05	5.30	4.92	4.65	4.40	4.25	4.10	3.95	3.82
9	8.32	7.65	7.25	7.00	6.75	6.55	6.40	6.30	6.15	6.10	5.35	5.00	4.70	4.49	4.30	4.15	4.00	3.90
10	8.35	7.75	7.35	7.05	6.80	6.60	6.45	6.35	6.20	6.14	5.40	5.02	4.80	4.52	4.35	4.19	4.05	3.92

(Contd.)...

Table 4.4 (Contd..)

	Values of r′													
t′	*0.1*	*0.2*	*0.3*	*0.4*	*0.5*	*0.6*	*0.7*	*0.8*	*0.9*	*1*	*2*	*3*	*4*	*5*
0.01	0.093	0.0430	0.0264	0.0180	0.0132	0.0100	0.0078	0.0062	0.0049	0.0040	0.00057	0.00015		
0.02	0.187	0.0865	0.0530	0.0365	0.0268	0.0205	0.0160	0.0125	0.0100	0.0081	0.00118	0.00020		
0.03	0.278	0.130	0.0800	0.0550	0.0405	0.0310	0.0240	0.0190	0.0150	0.0122	0.00184	0.00032		
0.04	0.368	0.174	0.107	0.0735	0.0540	0.0415	0.0322	0.0255	0.0202	0.0165	0.00244	0.00043		
0.05	0.450	0.215	0.133	0.0920	0.0675	0.0520	0.0400	0.0320	0.0255	0.0206	0.00305	0.00055		
0.06	0.530	0.257	0.160	0.110	0.0810	0.0610	0.0478	0.0380	0.0305	0.0250	0.00365	0.00065		
0.07	0.610	0.298	0.186	0.130	0.0950	0.0725	0.0565	0.0450	0.0360	0.0292	0.00430	0.00078		
0.08	0.680	0.340	0.214	0.148	0.108	0.0825	0.0645	0.0510	0.0412	0.0336	0.00500	0.00090		
0.09	0.750	0.378	0.236	0.164	0.122	0.0930	0.0730	0.0585	0.0470	0.0380	0.00570	0.00105		
0.1	0.815	0.415	0.260	0.180	0.134	0.103	0.0805	0.0640	0.0515	0.0420	0.00635	0.00118		
0.2	1.32	0.750	0.500	0.359	0.268	0.208	0.165	0.132	0.107	0.0880	0.0145	0.00278		
0.3	1.64	1.02	0.700	0.515	0.392	0.308	0.246	0.200	0.164	0.135	0.0238	0.00490		
0.4	1.86	1.22	0.870	0.650	0.510	0.405	0.328	0.268	0.220	0.182	0.0350	0.00750	0.00160	0.00038
0.5	2.03	1.37	1.00	0.770	0.610	0.490	0.400	0.330	0.275	0.230	0.0450	0.0104	0.00240	0.00056
0.6	2.16	1.49	1.12	0.875	0.700	0.570	0.468	0.390	0.325	0.276	0.0580	0.0138	0.00320	0.00080
0.7	2.28	1.60	1.22	0.965	0.775	0.640	0.525	0.445	0.375	0.320	0.0715	0.0175	0.00425	0.00108
0.8	2.36	1.69	1.30	1.04	0.850	0.715	0.600	0.500	0.425	0.364	0.0840	0.0212	0.00525	0.00140
0.9	2.45	1.75	1.38	1.11	0.920	0.775	0.650	0.550	0.475	0.404	0.0980	0.0260	0.00630	0.00165
1	2.54	1.85	1.45	1.18	0.975	0.825	0.700	0.595	0.510	0.444	0.113	0.0310	0.00840	0.00235
2	2.97	2.29	1.88	1.60	1.38	1.22	1.07	0.950	0.840	0.750	0.259	0.0950	0.0330	0.0115
3	3.20	2.50	2.10	1.82	1.60	1.42	1.28	1.15	1.05	0.960	0.388	0.165	0.0700	0.0275
4	3.36	2.66	2.25	1.97	1.75	1.58	1.42	1.30	1.20	1.10	0.495	0.235	0.112	0.0535
5	3.49	2.78	2.38	2.09	1.87	1.69	1.54	1.42	1.30	1.21	0.580	0.300	0.150	0.0715
6	3.59	2.90	2.47	2.18	1.95	1.78	1.65	1.52	1.40	1.30	0.660	0.360	0.195	0.0990
7	3.66	2.96	2.55	2.25	2.04	1.85	1.70	1.58	1.48	1.38	0.730	0.415	0.230	0.125
8	3.74	3.00	2.60	2.32	2.11	1.94	1.79	1.66	1.55	1.44	0.790	0.465	0.272	0.155
9	3.80	3.09	2.67	2.39	2.17	2.00	1.85	1.72	1.60	1.50	0.850	0.515	0.307	0.182
10	3.84	3.12	2.74	2.45	2.24	2.05	1.90	1.77	1.65	1.55	0.890	0.550	0.340	0.210

$$u = \frac{(350)^2 \times 4 \times 10^{-4}}{4 \times 0.145 \times 24 \times 60} = 5.87 \times 10^{-2}$$

$\therefore \quad W(u) = 2.316$

and $\quad s = \dfrac{1500/(24 \times 60)}{4 \pi \times 0.145} \times 2.316$

$= 1.32$ m.

Well Interference

If the zone of influence of two adjacent wells overlap (i.e., the wells are spaced at distances smaller than the sum of their radii of influence), the wells affect each other's drawdown and discharge. This effect is due to what is known as well interference. As a result of well interference, even though the total output (i.e., the discharge) of a multiple well system increases, the efficiency of each well (measured in terms of the discharge per unit drawdown) of the system decreases. Since the equation of flow in a confined aquifer is a linear one, one can use the principle of superposition to obtain the resulting drawdown at a point in a well field in which number of wells are being pumped simultaneously. This means, if s_i is the total drawdown at i^{th} observation well on acount of pumping of N wells located in the well field, then

$$s_i = \sum_{j=1}^{N} s_{ij} \qquad (4.30)$$

in which, s_{ij} is the drawdown at i^{th} observation well on account of pumping of j^{th} well as if there were no interference effects.

Considering steady flow conditions for two wells in a confined aquifer located distance B apart, the drawdown in the two wells s_{w1} and s_{w2} can be expressed as

$$s_{w1} = \frac{Q_1}{2\pi T} \ln \frac{r_0}{r_w} + \frac{Q_2}{2\pi T} \ln \frac{r_0}{B}, \qquad (4.31)$$

$$s_{w2} = \frac{Q_1}{2\pi T} \ln \frac{r_0}{B} + \frac{Q_2}{2\pi T} \ln \frac{r_0}{r_w} \qquad (4.32)$$

If $Q_1 = Q_2 = Q$, then

$$s_{w1} = s_{w2} = h_0 - h_w = \frac{Q}{2\pi T} \ln \frac{r_0^2}{Br_w}$$

This means,

$$Q = \frac{2\pi T (h_0 - h_w)}{\ln \dfrac{r_0^2}{Br_w}} \qquad (4.33)$$

Since, $\ln \dfrac{r_0^2}{Br_w} > \ln \dfrac{r_0}{r_w}$, it is obvious that the efficiency of an individual well has reduced. On the other hand, Eq. (4.33)can also be written as

$$\frac{2\,Q}{h_0 - h_w} = \frac{2\,\pi T}{\ln\left[r_0 / (B r_w)^{0.5}\right]} \qquad (4.34)$$

Since $(Br_w)^{0.5} >> r_w$, the value of $(h_0 - h_w)$ is relatively less for a discharge of 2 Q compared to the value of $(h_0 - h_w)$ when only a single well were to pump a discharge of 2 Q. This shows that the efficiency of a multiple well system is higher compared to that of a single well. But,
the efficiency of an individual well in a multiple well system is reduced.

Wells Near Aquifer Boundaries

The equations for radial flow towards well assume infinite extent of aquifer. However, in practice, there would be situations when a well may be located near hydrogeologic boundaries and the derived equations would not be applicable as such. The influence of such boundaries on ground water movement can be determined by the image well method. The image well method assumes straight line boundaries and replaces the real bounded field of flow with a fictitious field of flow with simple boundary conditions such that the flow patterns in the two cases are the same. Consider a pumping well located in the vicinity of a stream (i.e., recharge or permeable boundary). Obviously, the drawdown at the stream on account of pumping well would be zero. This real flow system is now assumed to be replaced with a fictitious flow system, Fig. 4.5. In addition to the real pumping well, the fictitious flow system has, in place of the boundary, an image well (which is a recharging one i.e., the one which pumps water into the aquifer) with the same capacity as that of the real well but located across the real boundary on a perpendicular thereto and at the same distance as the real well from the boundary. Obviously, this fictitious system would result in zero drawdown at the location of the boundary. This means that the flow condition of the real flow system is satisfied by the flow condition of the fictitious flow system. If the boundary is a barrier (i.e., impermeable) boundary, the method remains the same but the image well is also a pumping well. It should be noted that in the fictitious system the real and image wells operate simultaneously and the drawdowns can be obtained by considering the fictitious system as a multiple well system. When an aquifer is delimited by two or more boundaries, the effect of the other boundaries on each of the image wells is also to be considered. As a result, there would be several images, Fig.
When the image wells are too far from the region of interest, their influence on the flow system in the region of interest is negligible and are, therefore, not included in the computations.

Pumping well Perennial stream

G.S.

H Aquifer

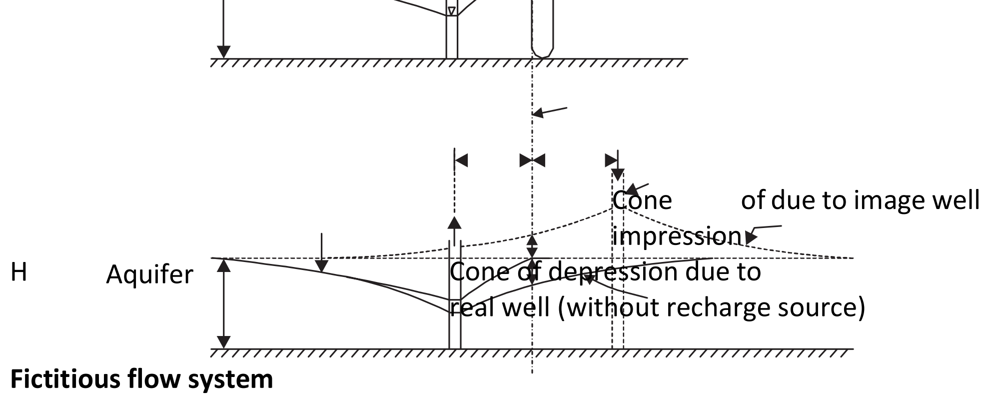
Impermeable

Real flow system Image plane
(Zero drawdown boundary)

X X $S_x S_x$ Q

Recharging image well

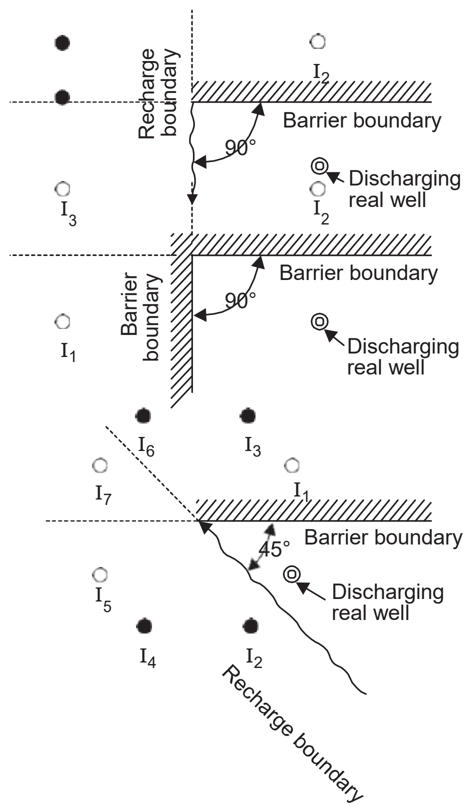
Actual cone of depression Q



Fictitious flow system

Fig. 4.5 Simulation of recharge boundary

I3
I1



Discharging image wellRecharging image well

Fig. 4.6 Image well system for different pairs of boundaries

GROUND WATER EXPLORATION

It is known that everywhere on the earth there is some water under the surface. Ground water planners, however, need to know whether the conditions of the available ground water would permit its economic withdrawal through wells. The purpose of ground water exploration is to delineate the water-bearing formations, estimate their hydrogeologic characteristics and determine the quality of water present in these formations. Some of the exploration methodsare briefly discussed in the following paragraphs.

Remote Sensing

Aerial photography, imaging (infra-red and radar) and low frequency electromagnetic aerial methods are included in the ‘‘remote sensing’’ methods of ground water exploration.

Valuable information associated with precipitation, evapotranspiration, interception, infiltration, and runoff can be inferred from aerial photographs by mapping the water area, geology and soil types, seepage areas, vegetation cover, and many other features (10). Satellite photographs can also be used for this purpose.

Recent developments in the nonvisible portion of the electromagnetic spectrum have resulted in several imaging techniques which are capable of mapping earth resources. Infrared imagery is sensitive to the differential head capacity of the ground and can map soil moisture, ground water movement, and faults (11). Radar imagery works in the 0.01–3 m wavelength range and can penetrate vegetation cover to provide subsurface information, such as soil moisture at shallow depths (12).

Buried subsurface channels and salt water intrusion fronts can be successfully located by using recently developed aerial electromagnetic exploration methods which operate in the frequency range of 3.0 to 9 kHz (13).

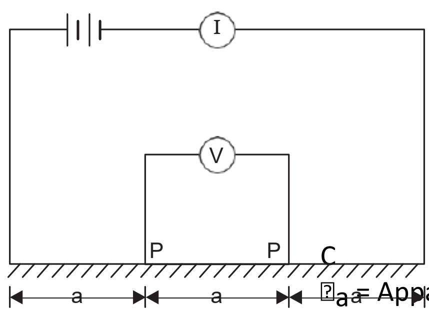
Surface Geophysical Methods

Surface geophysical methods reveal specific details of the physical characteristics of the local subsurface environment. This information can be interpreted suitably for the purpose of delineating the pre-glacial drainage pattern, mapping the location and extent of buried permeable deposits, direct exploration for ground water, and mapping of freshwater and salt water contact (14). The electrical resistivity method and seismic refraction method are the surface geophysical methods commonly used for ground water exploration.

Electrical Resistivity Method

The electrical resistivity of a rock depends on porosity, salinity of the fluid in the pore spaces, straightness or tortuosity of the interconnected pore spaces, presence of solid conductors,

$\rho_a = 2\pi a \frac{V}{I}$



C

C

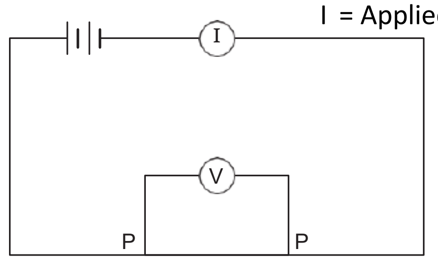
ρ_a = Apparent resistivity

V = Voltage difference between potential electrodes (P)

I = Applied current

Wenner arrangement

$\rho_a = \pi \frac{(L/2)^2 - (b/2)^2}{b} \frac{V}{I}$



C

C

a

b/2

L/2

b/2

L/2

Schlumberger arrangement

Fig. 4.7 Electrode arrays for electrical resistivity method such as clays or metallic minerals, and temperature (2). In the electrical resistivity method, electrical current is injected into the ground through two metal stakes (electrodes) and the resulting voltage between two other metal stakes is measured. The depth of measurement isdecided by the distance and the arrangement pattern of the four electrodes (Fig. 4.7) and the standard calibration curves. The changes in the electrical resistance of different earth layers are thus determined. Table 4.5 lists a typical order of values of resistivity for some common soils. Using the table and the plot of electrical resistivity versus depth, one can determine the type of subsurface layers at different depths. The electrical resistivity would vary with the salinity of the water included in the pores of earth material. Therefore, one should be careful in interpreting the results. It is advisable to prepare tables, similar to Table 4.5, or histograms of the resistivity for different regions and use these for the interpretation of resistivity measurements.

Table 4.5 Typical values of electrical resistivity for some soils (6)

Earth material	Electrical resistivity (ohm-metres)	
Clay	1	– 100
Loam	4	– 40
Clayey soil	100	– 380
Sandy soil	400	– 4000
Loose sand	1000	– 180,000
River sand and gravel	100	– 4000
Chalk	4	– 100
Limestones	40	– 3000
Sandstones	20	– 20,000
Basalt	200	– 1000
Crystalline rocks	10^3	– 10^6

Seismic Refraction Method

This geophysical method employs seismic waves to determine variations in the thickness of the unconfined aquifer and the zone where the most permeable strata are likely to exist. The method is based on the velocity variation of artificially generated seismic waves in the ground. Seismic waves are generated either by hammering on a metal plate, or by dropping a heavy ball, or by using explosives. The time between the initiation of a seismic wave on the ground and its first arrival at a detector (seismometre) placed on the ground is measured. For the seismic refraction method, one is interested only in the arrival of the critically refracted ray, i.e. the ray which encounters the boundary at such an angle that when it refracts in the lower medium, it travels parallel to the boundary at a higher velocity (2). The critically refracted ray travelling along the boundary radiates wavefronts in all directions and some of which return to the surface (Fig. 4.8). Using the appropriate formulas and the time-distance graph, one can determine the depth of the bedrock. Some representative values of refracted seismic wave velocities in different soils are given in Table 4.6. This method is more precise than the electrical resistivity method in the determination of the depth to bedrock (2). The depth of

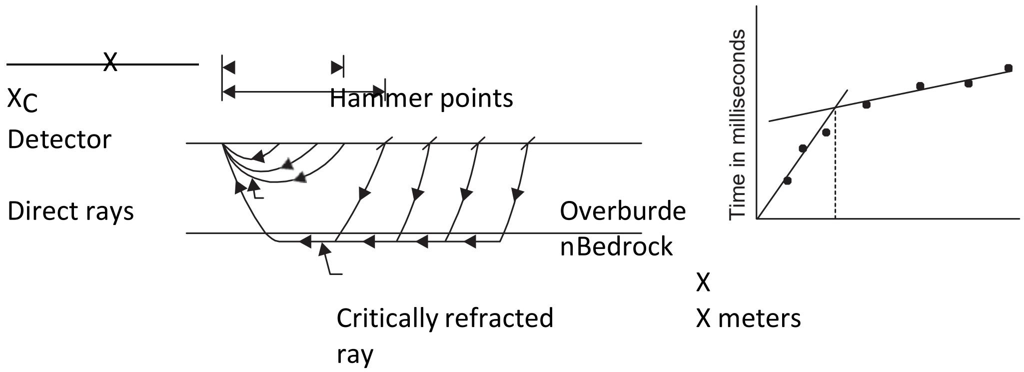

X
XC
Detector
Hammer points
Direct rays
Overburde
nBedrock
Critically refracted
ray
Time in milliseconds
X
X meters

Fig. 4.8 Seismic-refracted rays and time-distance graph

Table 4.6 Representative values of velocity of seismic refracted waves in some soils (15)

Material	Velocity (m/s)
Gravel, rubble or dry sand	457–915
Wet sand	610–1830
Clay	915–2740
Water (depnding on temperature and salinity)	1430–1680
Sea water	1460–1520
Sandstone	1830–3960
Shale	2740–4270
Chalk	1830–3960
Limestone	2130–6100
Salt	4270–5180
Granite	4570–5790
Metamorphic rocks	3050–7010

water table in sand gravel formation can also be determined accurately because of the sudden change in seismic velocity at the water table. One important requirement for the seismic refraction method to give accurate results is that the formations must be successively denser with increasing depths.

Well Logging Methods

Surface methods of ground water exploration do not give exact quantitative information about the subsurface environment. Quantitative information about subsurface strata can only be obtained by subsurface investigations which are conducted by personnel working on the surface and the equipment being lowered underground. The equipment extending into the ground measures one of several geophysical quantities, such as electrical resistivity, self- potential, temperature, gamma rays, and so on. Based on these measurements, well logs are prepared. For obtaining electrical resistivity log, one or more electrodes suspended on a conductor cable are lowered into a borehole filled with drilling fluid (6). An electric current is passed between these electrodes and other electrodes placed on the ground. The logging instrument measures the resistance to a flow of current between the electrodes. Thus the electrical resistivity is measured at different depths. The resistivity of any stratum depends primarily on its characteristics and the mineral content of water contained in the stratum.

Self potentials (or spontaneous potentials) are naturally-occurring electrical potentials which result from chemical and physical changes at the contacts between different types of subsurface geologic materials (6). For measuring the self potential at any depth, an electrode is lowered into an uncased borehole filled with drilling fluid by means of an electric cable connected to one end of a millivoltmeter. The other end of this millivoltmeter is connected to a ground terminal at the surface which is usually placed in a mud pit. No external source of current is required.

In gamma logging, natural radiation coming from different strata encountered in the borehole is measured. Such a log can yield qualitative information about subsurface strata.

Test Drilling

All geophysical exploration methods – surface as well as subsurface – and remote sensing methods are quicker and economic but yield results which may be interpreted in more than one way. Test drilling, however, provides the most positive information about the subsurface conditions. Test drilling can predict the true geohydrologic character of subsurface formations by drilling through them, obtaining samples, recording geologic logs, and conducting aquifer tests (2). The following data are usually obtained in test drilling:

Identification, location, and elevation of the site of each hole,

Geologic log of the strata penetrated,

Representative samples of strata penetrated,

Depth to static water level in each permeable stratum, and

Water quality samples and aquifer test data from water-bearing formations.

Rotary drilling and cable tool drilling are commonly used methods of drilling wells. The rotary drilling method is fast and is the most economical method of drilling wells in unconsolidated formations. However, accurate logging of cuttings is relatively difficult and the depth of water level cannot be predicted accurately unless electric logs have been taken for this purpose. Care should be taken in distinguishing between valid cuttings carried in mud suspension and the cuttings which have been delayed in reaching the surface. Cable tool drilling is suitable for drilling to moderate depths. Sampling of geologic materials is relatively more accurate and presents fewer difficulties. Cable tool drilling is, however, a more time-consuming method.

A good lithologic well log presents variation in the geohydrologic character of subsurface formation with depth and also the depth of the water table.

PUMPING TESTS (Or AQUIFER TESTS)

Aquifer characteristics and its performance can be best described by its hydraulic conductivity, transmissivity, and storativity. These quantities can be determined by analysing the data collected during aquifer tests or pumping tests. Measurements during an aquifer test include water levels at observation wells (before the start of pumping, at intervals during pumping, and for some time after pumping), the discharge rate, and the time of any variation in the discharge rate.

If the observations correspond to equilibrium conditions, one can use Eq. (4.16) for confined aquifers and Eq. (4.21) for unconfined aquifers to determine the hydraulic conductivity. Thus, for two observation wells located at distances r_1 and r_2 ($r_2 > r_1$) from the pumping well, Eq. (4.16) yields

$$K = -\frac{Q \log (r_2/r_1)}{2.73\, B\, (h_2 - h_1)} \tag{4.35}$$

in which, Q is negative for the pumping well. Similarly, Eq. (4.21) would yield

$$K = -\frac{Q \log (r_2/r_1)}{1.366\, (H_2^2 - H_1^2)} \tag{4.36}$$

For non-equilibrium conditions in confined aquifer, Eq. (4.28) would yield

$$T = \frac{0.183\, Q}{(s_2 - s_1)} \log \frac{t_2}{t_1} \tag{4.37}$$

Here, s_1 and s_2 are the drawdowns in an observation well (r distance away from the pumping well) at two different times t_1 and t_2 (from the beginning of pumping), respectively. If t_2 is chosen as 10 t_1 and $s_2 - s_1$ for this case be denoted by Δs, Eq. (4.37) is reduced to

$$T = \frac{0.183\, Q}{\Delta s} \tag{4.38}$$

Having known T, the storativity S can be determined from Eq. (4.28) by substituting suitable values of t and s obtained from the time-drawdown graph as illustrated in the following example.

Example 4.5 A well pumps water at a rate of 2500 m^3/day from a confined aquifer. Drawdown measurements in an observation well 120 m from the pumping well are as

Time since pump started in minutes	*Drawdown s in metres*
1	0.05
1.5	0.08
2	0.12
2.5	0.14
3	0.16
4	0.20
5	0.23
6	0.27
8	0.30
10	0.34
12	0.37

Time since pump started in minutes	*Drawdown s in metres*
14	0.40
18	0.44
24	0.48
30	0.52
50	0.61
60	0.64
80	0.68
100	0.73
120	0.76
150	0.80

follows:

Determine the aquifer characteristics S and T assuming that Eq. (4.28) is valid.

Solution:

From the time-drawdown graph (Fig. 4.9)

$\Delta s = 0.39$ m

Using Eq. (4.38),

$$T = \frac{0.183\ (2500)}{0.39} = 1173.1\ \text{m}^2/\text{day}$$

Substituting s = 0.74 m for t = 100 min = 0.07 day in Eq. (4.28)

$$0.73 = \frac{0.183\ (2500)}{1173.11} \log \frac{2.25\ (1173.1)\ (0.07)}{(120)^2\ S}$$

$\therefore \quad S = 1.72 \times 10^{-4}$

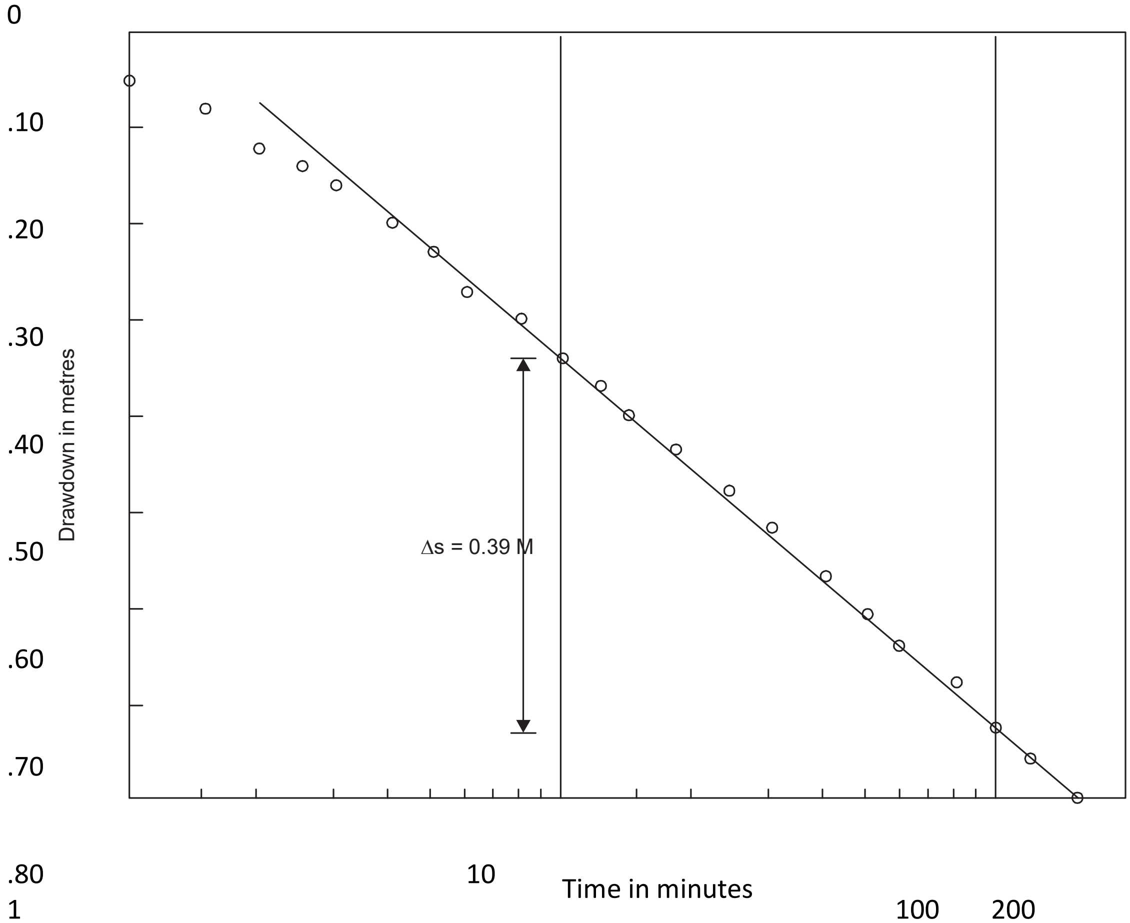


Fig. 4.9 Time-drawdown graph (Example 4.5)

DESIGN OF WATER WELLS

Well design is the process of specifying the physical materials and dimensions for various well components. The main objectives of well design are:

To obtain the highest yield with a minimum drawdown consistent with aquifer ca-pability and well requirement,

To obtain good quality water with proper protection from contamination,

To obtain sand-free water.

To ensure long life (30–40 years) of well, and

To have reasonable installation, maintenance, and operation costs.

The designer needs the following hydrogeologic information for making the design (6):

Stratigraphic information concerning the aquifer and overlying formations,

Transmissivity and storage coefficient of the aquifer,

The present and long-term water balance (i.e., inflow and outflow) conditions in the aquifer,

Grain size analyses of unconsolidated aquifer materials and identification of rocks and minerals, and

Water quality.

A water well has two main components – the casing and the intake portion. The casing serves as a vertical conduit for water flowing upward and also houses the pumping equipment. Some of the borehole length may, however, be left uncased if the well is constructed in consolidated rock. The intake portion in unconsolidated and semi-consolidated aquifers is usually screened. The well screen prevents fine aquifer material from entering the well with water and also serves to retain the loose formation material. In consolidated rock aquifer, the intake portion of the well may simply be an open borehole drilled into the aquifer.

Standard design procedure for a water well involves the following steps:

Selection of strata to be screened,

Design of well casing and housing pipe, and

Design of well screen.

Before starting a well design project it is worthwhile for the designer to study the design, construction, and maintenance of other wells in the area. The design practices may vary in different regions because of the hydro geologic conditions.

Selection of Strata to be Screened

The samples collected during drilling are sieve-analyzed and a litho logic well log is prepared. This log describes the characteristics (type of material, size distribution, values of d_{10} or d_{17}, d_{50}, d_{60}, etc., and uniformity coefficient d_{60}/d_{10}) of different subsurface strata. The litho logic log helps determine the thickness and permeability of each aquifer. The aquifers to be screened are thus decided.

Design of Well Casing and Housing Pipe

The well casing should meet the following requirements (16):

It should have a smooth exterior to minimise frictional resistance between the cas ing and the subsurface formations.

It should be of adequate size to permit the passage of drilling tools, operation of well development equipment, and installation of pumps. Its size must also assure the uphole velocity of 1.5 m/s or less so that the head loss is small.

The walls of the casing pipe must be of sufficient thickness and suitable material to resist stresses and corrosive action of ground water environment. The life of the casing pipe should be about 30 to 40 years after its installation. Cupronickel alloys, copper-bearing steel, stainless steel, P.V.C. pipes and fibre glass-reinforced epoxy pipes are the desirable types for casing material.

The field joints of the casing pipe must be leak-proof and have adequate strength.
The casing pipe, when used as a housing pipe, should have sufficiently large diameter at the housing elevation to accommodate the pump with enough clearance for its installation and operation. The housing pipe should have its diameter at least 5.0 cm greater than the nominal diameter of the pump and is set a few metres below the lowest drawdown level taking into account seasonal fluctuations and future development of ground water in the area. Table 4.7 presents recommended sizes of casing (i.e., well diameter) for different well yields.

Table 4.7 Recommended well diameters for different pumping rates

Anticipated pumping rate (m^3/day)	Nominal size of pump bowls (mm)	Optimum size of well casing (mm)	Smallest size of well casing (mm)
Less than 540	102	152 ID	127 ID
410– 950	127	203 ID	152 ID
820– 1910	152	254 ID	203 ID
1640– 3820	203	305 ID	254 ID
2730– 5450	254	356 OD	305 OD
4360– 9810	305	406 OD	356 OD
6540– 16400	356	508 OD	406 OD
10900– 20700	406	610 OD	508 OD
16400– 32700	508	762 OD	610 OD

Design of Well Screen

The design of a well screen (i.e., its length, slot, open area, diameter, and material) is the most important aspect of a well design. The basic requirements of a well screen are as follows (16):

It should be corrosion resistant,
It should be strong enough to prevent collapse,
It should prevent excessive movement of sand into the well, and
It should have minimum resistance to the flow of water into the well.

Length of Well Screen

The intake portion of a well must, obviously, be placed in the zones of the maximum hydraulic conductivity. Such zones are determined by interpreting the lithologic log, visual inspection and sieve analysis of the samples collected during drilling, laboratory tests for hydraulic conductivity and the results of pumping tests. The optimum length of the well screen depends primarily on the nature of the aquifer stratification and the permissible drawdown.

In the case of a homogeneous unconfined aquifer of thickness less than 45 m the screening of the bottom one-third to one-half of the aquifer is recommended (6). In thick and deep aquifers, however, as much as 80 per cent of the aquifer may be screened to obtain a higher specific capacity and greater efficiency even though the resulting yield may be less. These guidelines are applicable to non-homogeneous unconfined aquifers also. However, screen sections are positioned in the most permeable layers of the lower portions of the aquifer (leaving depth of about 0.3 m at the upper and lower ends of the screen to prevent finer material of the transition zone from moving into the well) so that maximum drawdown is available. Wherever possible, the total screen length should be approximately one-third of aquifer thickness.

For homogeneous confined aquifers, the central 80 to 90 percent of the aquifer thickness should be screened assuming that the water level in the well would always be above the

upper boundary of the aquifer. In case of non-homogeneous confined aquifer, 80 to 90 per cent of the most permeable aquifer layers should be screened.
If the effective size of two strata are the same, the stratum with lower uniformity coefficient (i.e., relatively poorly graded) is more permeable and should, therefore, be screened.

Well-Screen Slot Openings

Well screen slot openings primarily depend on the size distribution of the aquifer material and also on whether the well is naturally developed or filter-packed (i.e., artificially gravel-packed). Wells in aquifers with coarse-grained (d_{10} > 0.25 mm) and non-homogeneous material can be developed naturally. But wells in aquifers with fine-grained and homogeneous material are best developed using a filter pack (or gravel pack) outside the well screen.
In a naturally developed well, the screen slot size is selected so that most of the finer aquifer materials in the vicinity of the borehole are brought into the screen and pumped from the well during development. The process creates a zone of graded formation materials extending 0.3 to 0.6 m outward from the screen (6). The slot size for the screen of such wells can be selected from Table 4.8.

Table 4.8 Selection of slot size for well screen (17)

Uniformity coefficient of the aquifer being tapped	Condition of the overlying material	Slot size in terms of aquifer material size
> 6	fairly firm; would not easily cave in	d_{70}
> 6	soft; would easily cave in	d_{50}
= 3	fairly firm; would not easily cave in	d_{60}
= 3	soft; would easily cave in	d_{40}

If more than one aquifer is tapped, and the average size of the coarsest aquifer is less than four times the average size of the finest aquifer, the slot size should correspond with the finest aquifer. Otherwise, slot size must vary and correspond with the sizes of aquifer material (16). A more conservative slot size should be selected if: (i) there is some doubt about the reliability of the samples, (ii) the aquifer is thin and overlain by fine-grained loose material,
the development time is at a premium, and (iv) the formation is well-sorted. Under these conditions, slot sizes which will retain 40 to 50 per cent of the aquifer material (i.e., d_{60} to d_{50}) should be preferred (6).
In filter-packed wells, the zone in the immediate vicinity of the well screen is made more permeable by removing some formation material and replacing it with specially graded material. This filter pack or gravel pack separates the screen from the aquifer material and increases the effective hydraulic diameter of the well. A filter pack is so designed that it is capable of retaining 90 per cent of the aquifer material after development. Well screen openings should be such that they can retain 90 per cent of the filter pack material (6). The filter pack material must be well graded to yield a highly porous and permeable zone around the well screen. The uniformity coefficient of the filter pack should be 2.0 or less so that there is less segregation during placing and lower head loss through the pack. The filter pack material should be clean and well-rounded. Clean material requires less development time and also results in little loss of material during development. Well-rounded grains make the filter pack more permeable which reduces drawdown and increases the yield. Further, the filter pack must contain 90 to 95% quartz grains so that there is no loss of volume caused by the dissolution of materials. For minimum head loss through the filter pack and

minimum sand movement, the pack-aquifer ratio (i.e., the ratio of the average size of the filter pack material to the average size of the aquifer material) should be as follows (16):
Pack-aquifer Ratio
Uniform aquifer with uniform filter pack 9–12.5
Non-uniform aquifer with uniform filter pack 11–15.5 The thickness of the filter pack designed in this manner should be between 15 and
20 cm.

Open Area of Well-Screen

For head loss through the well screen to be minimum, Peterson et al. (18) have suggested that the value of the parameter C_c A_p L/D should be greater than 0.53. Here, C_c is the coefficient of contraction for the openings, A_p the ratio of the open area to the total surface area of the screen, L the screen length, and D is the diameter of well screen. Theoretical studies conducted at the UP Irrigation Research Institute have shown that the parameter C_c C_v A_p L/D should be greater than 1.77. Here, C_v is the coefficient of velocity. A factor of safety of 2.5 is further recommended by Sharma and Chawla (16). This means that C_c C_v A_p L/D should be greater than 4.42.

Diameter of Well Screen

The screen diameter should be such that there is enough open area so that the entrance velocity of water generally does not exceed the design standard of 3 cm/s (6). Table 4.9 gives values of the optimum diameter for different values of the yield and hydraulic conductivity of the aquifer. These values have been worked out considering the cost of screens, cost of boring and the running expenses (16). USBR's recommended values have also been given in Table 4.9.

Table 4.9 Optimum diameter of well screen (16, 19)

Well discharge (m^3/s)	Optimum diameter of well screen in cm for hydraulic conductivity equal to			USBR's recommended value of well screen diameter(cm)
	0.04 cm/s	0.09 cm/s	0.16 cm/s	
0.04	15	18	22	25
0.08	20	25	30	30
0.12	23	28	33	35
0.16	25	30	35	40

Entrance Velocity

The entrance velocity of water moving into the well screen should be kept below a permissible value which would avoid movement of fine particles from the aquifer and filter pack to the well. The permissible entrance velocity depends on the size distribution and the granular structure of the aquifer material, the chemical properties of the ground water and shape of the screen openings. Its exact evaluation is difficult. The permissible entrance velocity is usually taken as 3 cm/s for the design of the well screen.

Well Screen Material

Four factors govern the choice of material used for the construction of a well screen. These are:
water quality, (ii) presence of iron, (iii) strength requirements of screen, and (iv) cost of screen. Quality analysis of ground water usually shows that the water is either corrosive or incrusting. Corrosive water is usually acidic and contains dissolved oxygen and carbon dioxide

which accelerate the corrosion. Corrosion may further increase due to higher entrance velocities. Incrustation is caused due to precipitation of iron and manganese hydroxides and other materials from water. It is, therefore, important to use corrosion-resistant materials for the fabrication of a well screen. The following alloys (in decreasing order of their ability to resist corrosion) or their suitable variations are used for the fabrication of well screens (16).

Monel alloy (or Monel metal) (70% nickel and 30% copper)

Cupro-nickel (30% nickel and 70% copper)

Everdur A alloy (96% copper, 3% silicon and 1% manganese)

Stainless steel (74% low carbon steel, 18% chromium and 8% nickel)

Silicon red brass (83% copper, 1% silicon and 16% zinc)

Anaconda brass (or Gilding metal) (85% copper and 15% zinc)

Common yellow brass (67% copper and 33% zinc)

Armco iron (99.84% pure iron)

Low carbon steel

Ordinary cast iron.

MEHODS OF WELL CONSTRUCTION

The operations involved in well construction are drilling, installing the casing, placing a well screen and filter pack, and developing the well to ensure maximum sand-free water yield. Shallow wells, generally less than about 15 m deep, are constructed by digging, boring, driving or jetting. Deep wells are constructed using drilling methods. Wells used for irrigation purposes are generally deep.

Digging

Wells in shallow and unconsolidated glacial and alluvial aquifers can be dug by hand using a pick and shovel. Loose material is brought to the surface in a container by means of rope and pulleys. The depth of a dug well may vary from about 3 to 15 m depending upon the position of the water table. Dug wells usually have large diameter ranging from about 1 to 5 m. Dug wells must penetrate about 4 to 6 m below the water table. The yield of the dug wells is generally small and is of the order of about 500 litres per minute.

Boring

Hand-operated or power-driven earth augers are used for boring a well in shallow and unconsolidated aquifers. A simple auger has a cutting edge at the bottom of a cylindrical container (or bucket). The auger bores into the ground with rotary motion. When the container is full of excavated material, it is raised and emptied. Hand-bored wells can be up to about 20 cm in diameter and about 15 m deep. Power-driven augers can bore holes up to about 1 m in diameter and 30 m deep.

Driving

In this method, a series of connected lengths of pipe are driven by repeated impacts into the ground to below the water table. Water enters the well through a screened cylindrical section which is protected during driving by a steel cone at the bottom. Driven wells can be installed only in an unconsolidated formation relatively free of cobbles or boulders. The diameters of driven wells are in the range of about 3–10 cm. Such wells can be constructed up to about 10 m, if hand driven, and up to about 15 m when heavy hammers of about 300 kg are used. The maximum yield of driven wells is usually around 200 litres per minute. The main advantage of a driven well is that it can be constructed in a short time, at minimum cost, and by one man.

Jetting

The jetting (or jet drilling) method uses a chisel-shaped bit attached to the lower end of a pipe string. Holes on each side of the bit serve as nozzles. Water jets through these nozzles keep the bit clean and help loosen the material being drilled. The fluid circulation system is similar to that of a direct rotary drilling method. With water circulation maintained, the drill rods and the bit are lifted and dropped in manner similar to cable tool drilling but with shorter strokes. Jet drilling is limited to drilling of about 10 cm diameter wells to depths of about 60 m, although larger diameter wells have been drilled up to about 300 m by this method (6). Other drilling methods have replaced jet drilling for deep and larger diameter wells.

Cable Tool Drilling

It is the earliest drilling method developed by the Chinese some 4000 years ago. A cable tool drilling equipment mainly consists of a drill bit, drill stem, drilling jars, swivel socket, and cable (Fig. 4.10). The cable tool drill bit is very heavy (about 1500 kg) and crushes all types of earth materials. The drill stem provides additional weight to the bit and its length helps in maintaining a straight vertical hole while drilling in hard rock. The length of the drill stem varies from about 2 to 10 m and its diameter from 5 to 15 cm. Its weight ranges from 50 to 1500 kg. Drilling jars consist of a pair of linked steel bars and help in loosening the tools when these stick in the hole. Under the normal tension of the drilling line, the jars are fully extended. When tools get stuck, the drilling line is slackened and then lifted upward. This causes an upward blow to the tools which are consequently released. The swivel socket (or rope socket) connects the string of tools to the cable. The wire cable (about 25 mm in diameter) which carries and rotates the drilling tool on each upstroke is called the drill line. The cable tool drilling rig mainly consists of a mast, a multiline hoist, a walking beam, and an engine. Drill cuttings are removed from the well by means of bailers having capacities of about 10 to 350 litres. A bailer is simply a pipe with a valve at the bottom and a ring at the top for attachment to the bailer line. The valve allows the cuttings to enter the bailer but prevents them from escaping. Another type of bailer is called the sand pump or suction bailer which is fitted with a plunger. An upward pull on the plunger produces a vacuum which opens the valve and sucks sand or slurried cuttings into the tubing. Most sand pumps are about 3 m long.

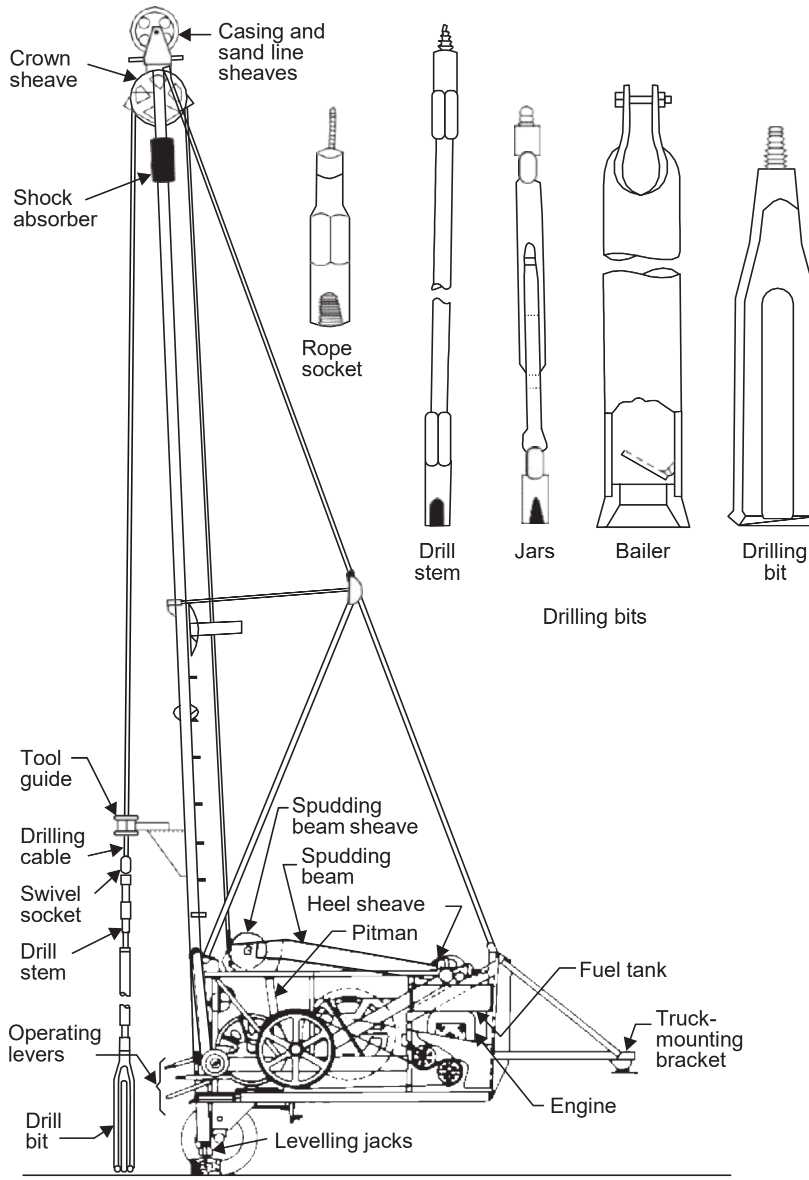


Fig. 4.10 Line sketch of a typical cable tool drilling rig (6)

While drilling through consolidated formations, most boreholes are drilled as ‘‘open hole’’,i.e., no casing is used during the drilling operation. In such conditions the cable tool bit is essentially a crusher. On the other hand, there is a danger of caving in while drilling through unconsolidated formations. For this reason, the casing pipe must follow the drill bit closely to keep the borehole open in unconsolidated formations. Also, in the case of such formations, the drilling action of the bit is primarily a loosening and mixing process. Actual crushing would take place only if a large stone or boulder were encountered.

For the driving operation of the casing pipe, a drive head is fitted to the top of the casing. The drive head serves as an anvil and protects the top of the casing. Similarly, a drive shoe made of hardened and tempered steel is attached to the lower end of the casing pipe. The shoe prevents the damage to the bottom end of the casing pipe when it is being driven. The casing is driven down by means of drive clamps, constructed of heavy steel forgings made in halves, fastened to the top of the drill stem. Drive clamps act as the hammer face and the up-and-down motion of tools provides the weight for striking the top of the casing pipe and thus driving it into the ground.

The procedure for drilling through unconsolidated formation consists of repeated driving, drilling, and bailing operations. The casing pipe is initially driven for about 1 to 3 m in the ground. The material within the casing pipe is then mixed with water by the drill bit to form slurry. The slurry is bailed out and the casing pipe is driven again. Sometimes, the hole is drilled 1 to 2 m below the casing pipe; the casing is then driven down to the undisturbed material and drilling is resumed. The drilling tools make 40 to 60 strokes of about 40 to 100 cm length every minute. The drill line is rotated during drilling so that the resulting borehole is round. The slurry formed by the mixing of cuttings with added water (if not encountered in the ground) reduces the friction on the cutting bit and helps in bailing operations.

If the friction on the outside of the casing pipe increases so much that it cannot be driven any more or if further driving might damage the pipe, a string of smaller casing is inserted inside the first one. Drilling is thus continued. Sometimes, two or three such reductions may be required to reach the desired aquifer. The diameter of the well is reduced. If such a situation is anticipated, the casing in the upper part should be of larger diameter. The drilling process through consolidated formation, not requiring casing, would consist of repeated drilling and bailing operations only.

The cable tool method has survived for thousands of years mainly because of its suitability in a wide variety of geological conditions. It offers the following advantages (6):

Cable tool drilling rigs are relatively cheaper.

The rigs are simpler and do not require sophisticated maintenance.

The machines have low power requirements.

The borehole is stable during the entire drilling operation.

Recovery of reliable samples is possible at every depth.

Wells can be drilled in water-scarce areas.

Because of their size, the machines can be operated in more rugged, inaccessible terrain or in other areas where limited space is available.

Wells can be drilled in formations where water is likely to be lost.

Slow driling rate, higher cost of casing pipe, and difficulty in pulling back long strings of casing pipes are some of the disadvantages of cable tool drilling.

Direct Rotary Drilling

Direct rotary drilling is the fastest method of drilling deep wells of diameters of up to 45 cm (or more with the use of reamers) through unconsolidated formations. The drilling bit is attached to a heavy drill pipe which is screwed to the end of the kelly which is a drill pipe of square section (Fig. 4.11). The drill collar or stabilizer helps in maintaining, straight hole in soft formations through its large wall contact. The drill pipe is turned by a rotating table which fits closely round the kelly and allows the drill rod to slide downward as the hole deepens. The drilling rig consists of a mast, a rotating table, a pump, a hoist, and an engine. The borehole is drilled by rotating a hollow bit attached to the lower end of a string of a drill pipe. Cuttings are

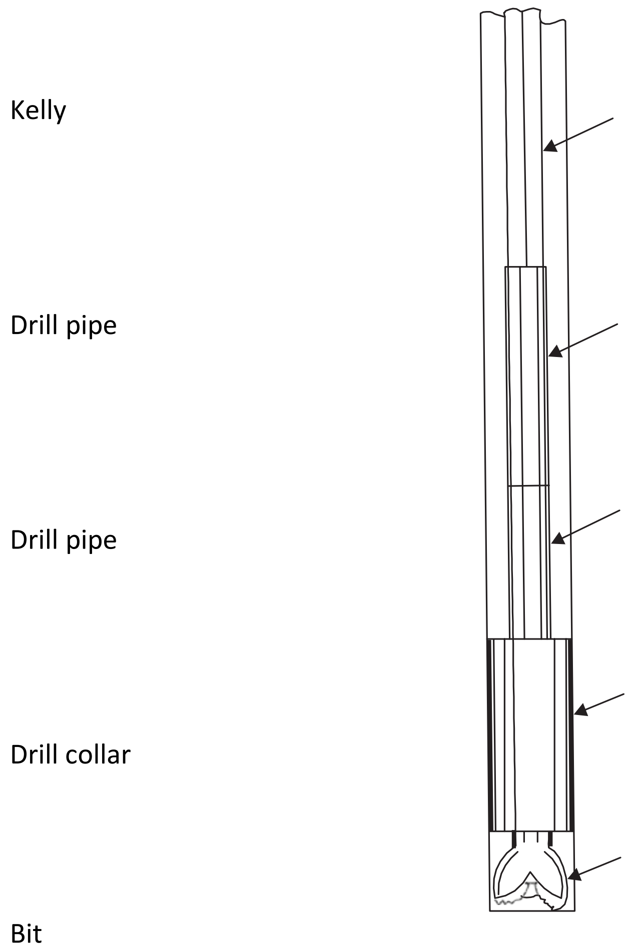


Fig. 4.11 Drill string for rotary drilling

removed continuously by pumping drilling fluid (a mixture of clay and water with some additives to make it viscous) down the drill pipe and through the orifices in the bit. The drilling fluid then flows upward through the annular space between the drill pipe and the borehole, carrying the cuttings in suspension to the surface settling pits where the cuttings settle down in the pits. The clear drilling fluid is pumped back into the borehole. The settling pits can either be portable or excavated for temporary use during drilling and then backfilled after completion of the well. Usually no casing is required during drilling because the drilling mud forms a clay lining on the borehole walls which prevents the formation materials from caving in. After drilling, the casing pipe with perforated sections opposite the aquifers is lowered into the borehole. The drilling rotary method has become the most common due to its following advantages (6):

Drilling rates are relatively high.

Minimum casing is required during drilling.

Rig mobilisation and demobilisation are fast.

Well screens can be set easily as part of the casing installation.

Some of the major disadvantages of the direct rotary method are as follows.

Drilling rigs are expensive.
It is costly to maintain them.
The mobility of the rigs is restricted depending on the slope and wetness of the land surface.
The collection of accurate samples requires special procedure.
The drilling fluid may cause the plugging of some aquifer formations.
Reverse Rotary Drilling
The direct rotary drilling method is capable of drilling boreholes with a maximum diameter of about 60 cm. High-capacity wells, particularly those with filter pack, need to be much larger in size. Besides, the drilling rate becomes smaller with increase in borehole diameter in the case of direct rotary drilling. To overcome these limitations, the reverse rotary drilling technique has been developed. This technique is capable of drilling boreholes of about 1.2 m diameter in unconsolidated formation. Recently, the reverse rotary method has been used in soft consolidated rocks such as sandstone, and even in hard rocks using both water and air as the drilling fluid.
In reverse rotary drilling, the flow of the drilling fluid is opposite to that in direct rotary drilling. The reverse rotary drilling rig is similar to the direct rotary drilling rig except that it requires larger-capacity centrifugal pumps, a larger diameter drill pipe, and other components also of larger size. The drilling fluid moves down the annular space between the borehole wall and the drill pipe, and picks up the cuttings before entering the drill pipe through the ports of the drill bit. The drilling fluid, along with its cuttings load, moves upwards inside the drill pipe which has been connected to the suction end of the centrifugal pump through the kelly and swivel. The mixture is brought to a settling pit where the cuttings settle at the bottom and the drilling fluid (i.e., muddy water) moves down the borehole again. The drilling fluid is usually water mixed only with fine-grained soil. The hydrostatic pressure and the velocity head of the drilling fluid moving down the borehole supports the borehole wall. To prevent the formation from caving in, the fluid level must always be up to the ground surface even when drilling is suspended temporarily. The advantages of the reverse rotary drilling method are as follows.
The formation near the borehole is relatively undisturbed compared to other meth- ods.
Large-diameter holes can be drilled rapidly and economically.
No casing is required during the drilling operation.
Well screens can be set easily while installing the casing.
The boreholes can be drilled through most geologic formations, except igneous and metamorphic rocks.
Because of the low velocity of the drilling fluid, there is a little possibility of its entering the formation.
The disadvantages of the reverse rotary drilling method are as follows (6):
A large quantity of water is needed.
The reverse rotary drilling rig is costlier because of larger size of equipment.
Large mud pits are required.
Some drill sites may be inaccessible because of the larger size of the rig.

WELL COMPLETION

After drilling a well, the well screen and filter pack (wherever necessary) are to be placed and the casing removed. If the formations are sufficiently strong and stable, ground water may directly enter the uncased well. In unconsolidated formations, however, a casing with perforation (or a well screen) is needed to support the outside material and also to admit water freely into the well.

The installation of the well screen and the removal of the casing is best done by the pull- back method in which the casing is installed to the full depth of the well and the well screen, whose size is smaller than that of casing, is lowered inside the casing. The casing is then pulled back or lifted far enough to expose the screen to the water-bearing formation (Fig. 4.12).

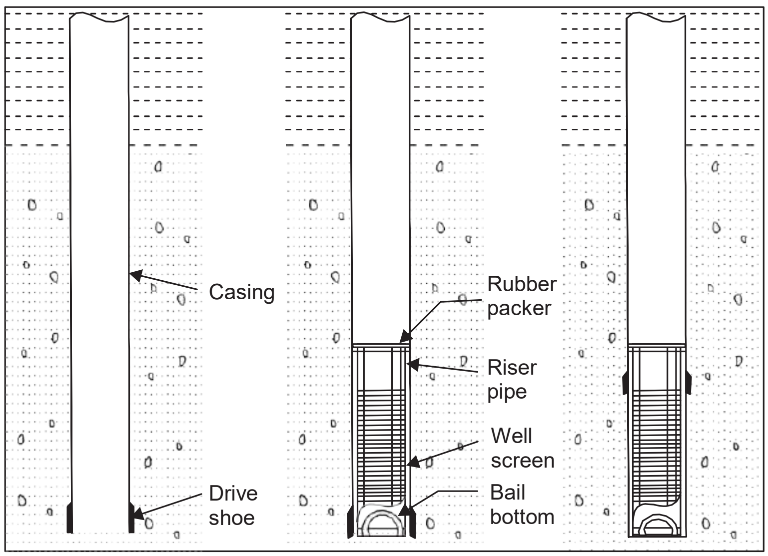


Fig. 4.12 Pull-back method of installing screen

In the case of rotary-drilled wells, setting the casing to the bottom of the hole and then pulling it back may appear to be extra and unnecessary work in view of the drilling fluid supporting the borehole wall. But this extra work prevents serious problems which may arise on account of premature caving in which may occur when the viscosity of the drilling fluid is reduced prior to development. This casing is also useful when there is likely to be a longer period between drilling and screen installation, and during which period a momentary loss of drilling fluid may cause partial collapse of the borehole.

A filter pack is generally placed in large-diameter wells by the reverse circulation of the fluid in the well as the filter pack material is fed into the annular space outside the screen by a continuous-feed hopper. When the filter pack material fills the space around the well screen, the transporting water is drawn upward through the screen openings (Fig. 4.13).

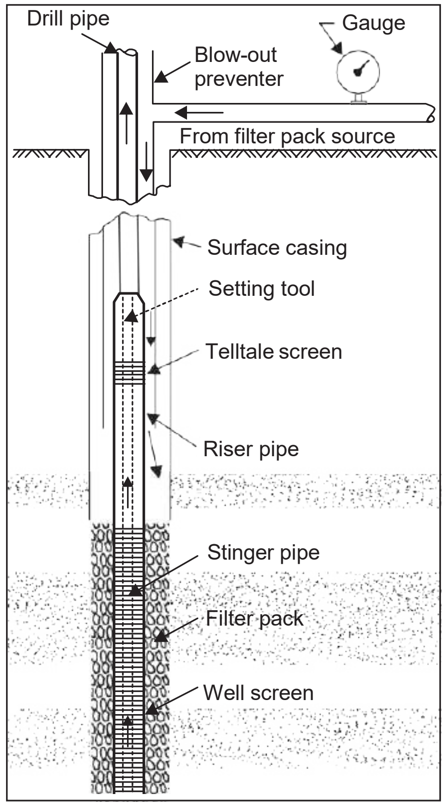


Fig. 4.13 Filter packing of wells

DEVELOPMENT OF WELLS

Drilling operations for well excavation change the hydraulic characteristics of the formationmaterials in the vicinity of the borehole. Very often, these changes result in the reduction of the hydraulic conductivity close to the borehole. When a well is drilled with a cable tool rig equipped with a casing driver, the repeated blows on the casing rearrange the grains in the vicinity of the casing. In rotary drilling methods, the drilling fluids containing clay may flow into the aquifer for some distance and thus plug the pore spaces of the permeable formation. Before commissioning the well for use, it is, therefore, necessary to repair the damage done tothe aquifer by the drilling operations. Besides, there is also a need to improve the basic physical characteristics of the aquifer in the vicinity of the well screen so that water can flow more freely into the well. A well is, therefore, 'developed' in order to attain these two objectives, and thus, maximise well yield. Well development involves applying some form of energy to the water-bearing formation in the vicinity of the well so as to remove fine materials (including drilling mud) from the aquifer and rearrange formation particles so that the well yields clearsand-free water in maximum quantity with minimum drawdown. Well development serves the following beneficial purposes :

It increases the permeability of the aquifer material surrounding the well and filter pack (if present) by:

reducing the compaction and intermixing of grains of different sizes during drill- ing by removing fine grains,

removing the filter cake or drilling fluid film that coats the borehole,
removing much or all of the drilling fluid which has entered the aquifer,
breaking sand-grain bridging across the screen openings, and
increasing the natural porosity of the previously undisturbed formation near the borehole by removing the finer fraction of the aquifer material.
It creates a graded zone of aquifer material around the screen in a naturally devel- oped well. This effect stabilises the formation so that the well will yield sand-free water.
It reduces the head loss near the well screen.
It increases the useful life of the well screen.
It brings the well to its maximum specific capacity, i.e., the maximum yield at mini- mum drawdown.
The methods usually adopted for well development are as follows (6):
Overpumping,
Backwashing.
Mechanical surging,
Air surging and pumping,
High-velocity jetting, and
High-velocity water jetting combined with simultaneous pumping.
There are several variations of most of these methods. Only the main features of these methods have been described in the following paragraphs.

Overpumping

In the ovrpumping method, the well is pumped at a discharge rate higher than the discharge rate of the well during its normal operation. The logic of the method is that any well which can be pumped sand-free at a high rate can be pumped sand-free at a lower rate. It is the simplest method of developing wells. However, the development by this method is not effective and the developed well is seldom efficient. The aquifer material is also not fully stabilised. This incomplete development is due to the following reasons:
Water flows in only one direction and some sand grains may be left in a bridged condition. The formation is thus partially stabilised.
Most of the development takes place in the most permeable zones of the aquifer which are usually closest to the top of the screen. Therefore, less development takes place in the lower layers of the aquifer.
Besides, this method generally uses the pump intended for regular use during the normal operation of the well. Pumping of silt-laden water at higher rates can reduce efficiency of the pump.

Backwashing

Reversal of flow through the screen openings agitates the aquifer material, removes the finer fraction and rearranges the remaining aquifer particles. These effects usually cause effective development of the well. The ‘‘rawhiding’’ method of backwashing consists of alternately lifting a column of water significantly above the pumping level and then letting the water fall back into the well. To minimise the changes of sand-locking the pump, its discharging rate should be gradually increased to the maximum capacity before stopping the pump. During this process, the well is occasionally pumped to waste to remove the sand brought to the well by the surging action of this method of well development. As in the case of the overpumping method, the surging action may be concentrated only in the upper layers of the aquifer. Besides, the surging effect is not vigorous enough to cause maximum benefits. When compared with other methods of well development, the overall effectiveness of backwashing as well as overpumping methods in case of high-capacity wells is rather limited.

Mechanical Surging

In this method, a close-fitting surge plunger, moving up and down in the well casing, forces water to flow into and out of the well screen. The initial movements of the plunger should be relatively gentle so that the material blocking the screen may go into suspension and then move into the well. To minimise the problem of the fine materials going back to the aquifer from the well, the fine material should be removed from the well as often as possible. The surging method is capable of breaking sand bridges and produces good results. However, it is not very effective in developing filter-packed wells because the water movement is confined only up to the filter pack and the aquifer remains unaffected by the surging action.

Air Surging and Pumping

This method requires two concentric pipes – the inner pipe known as the air line and the outer one known as the pumping pipe (or eductor pipe). The assembly of these pipes is lowered into the well. In air surging, compressed air is injected through air line into the well to force aerated water up through the annular space between the air line and the pumping pipe. As this aerated water reaches the top of the casing, the air supply is stopped so that aerated water column starts falling. Air-lift pump is used to pump the well periodically to remove the sand brought into the well as a result of air surging. The compressed air produces powerful surging action. This method is used to develop wells in consolidated and unconsolidated formations.

High-Velocity Jetting

This method consists of shooting out high-velocity jets of water from a jetting tool to the aquifer through the screen openings. The equipment of this method consists of a jetting tool provided with two or more equally spaced nozzles, high-pressure pump, high-pressure hose and connections, and a water supply source. The forceful action of high-velocity jets loosens the drilling mud and agitates, and rearranges the sand and gravel paritcles around the well. The loosened material is removed by pumping. In this method of development, the entire surface of the screen can be subjected to vigorous jet action by slowly rotating and gradually raising and lowering jetting tool.

This method has the following advantages:

The energy is concentrated over a small area with greater effectiveness.

Every part of the screen can be developed selectively.

The method is relatively simple.

The method of jetting is particularly successful in developing highly stratified and unconsolidated water-bearing formations.

High-Velocity Water Jetting Combined with Simultaneous Pumping The method of high-velocity water jetting results in very effective development of wells. But, maximum development efficiency can be obtained by combining high-velocity water jettingwith simultaneous air-lift pumping method (Fig. 4.14). The method requires that the volume

of water pumped from a well will always be more than that pumped into it so that the waterlevel in the well is always below the static level and there is a continuous movement of water from the aquifer to the well. This would help remove some of the suspended material loosened by the jetting operation .

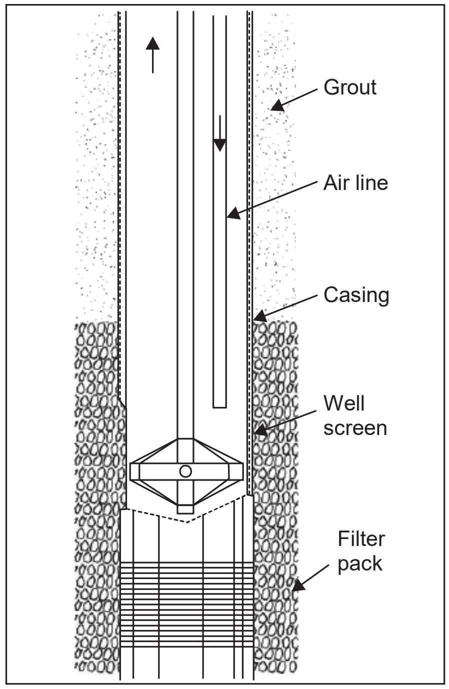


Fig. 4.14 Jetting and air-lift pumping

PUMPING EQUIPMENT FOR WATER WELLS

In most wells, the static water level is below the ground surface and, hence, flowing wells are rare. The water has to be lifted from inside the well to the ground surface. Rope and bucket with or without windlass have been used and are still used for shallow wells and for low discharges. For deeper wells and high yields of water, pumps have to be used.

The purpose of installing pumps in wells is to lift water from inside the well to the ground surface. Pumps can be broadly classified as shallow well pumps and deep well pumps depending upon the position of the pump and not the depth of the well. A shallow well pump is installed on the ground and lifts water from the well by suction lift. A deep well pump is installed within the well casing and its inlet is submerged below the pumping level. If the pumping level is lower than the limit of a suction lift (about 7.5 m), only the deep well pumpshould be used.

Pumps are also classified on the basis of their design as positive displacement pumps and variable displacement pumps (6). Positive displacement pumps discharge almost the same volume of water irrespective of the head against which they operate. The input power, however, varies in direct proportion to the head. Such pumps are used extensively in ground water monitoring wells, hand pump-equipped wells, and wind-powered wells. They are rarely used for large-capacity water wells. The most common type of positive displacement pumps is the piston pump.

The variable displacement pumps are used for large-capacity wells. For these pumps, there is an inverse relationship between the discharge and the working head. Maximum input power is required when the pump has to operate at low heads delivering large volumes of water. The major types of variable displacement pumps are as follows (6):

Centrifugal pumps:

suction lift pump,

deep-well turbine pump, and
submersible turbine pump.
Jet pumps
Air-lift pumps

Centrifugal Pumps

Centrifugal pumps are the most popular. They are capable of delivering large volumes of water against high as well as low head with good efficiency. Besides, these pumps are relatively simple and compact. The basic principle of centrifugal pumping can be understood by consideringthe effect of swinging a bucket of water around in a circle at the end of a rope. The centrifugal force causes the water to press against the bottom of the bucket rather than spill out of the bucket. If a hole is cut in the bottom, water would discharge through the opening at a velocity which would depend on the centrifugal force. If an airtight cover were put on the bucket top, a partial vacuum would be created inside the bucket as the water would leave through the opening in the bottom. If a water source is connected to the airtight cover through an intake pipe, the partial vacuum will draw additional water into the bucket as the water is being discharged through the bottom hole. The bucket and cover of this example correspond to the casing of a centrifugal pump; the discharge hole and the intake pipe correspond to the pump outlet and inlet, respectively; the arm that swings the bucket corresponds to the energy source and the rope performs the function of a pump impeller.

Suction-lift pumps create negative pressure at the pump intake. The atmospheric pressure at the free surface of water in the well forces the well water into and up the intake pipe. The maximum suction lift depends on the atmospheric pressure (10.4 m of water head), vapour pressure of water, head loss due to friction, and the head requirements of the pump itself. Under field conditions, the average suction-lift capability of a suction-lift centrifugal pump is about 7.5 m (6).

A deep-well vertical turbine pump consists of one or more impellers housed in a single- or multi-stage unit called a bowl assembly. Each stage gives a certain amount of lift and sufficient number of stages (or bowl assemblies) are assembled to meet the total head requirement of the system (6).

Vertical turbine pumps in high-capacity wells are highly reliable over long periods of time. The motors of these pumps are not susceptible to failure caused by fluctuations in electric supply. Motor repairs can be carried out easily because of their installation on the ground surface. These pumps, however, cannot be used in wells which are out of the alignment. Besides, these pumps require highly skilled personnel for installation and service.

Submersible pumps have bowl assemblies which are the same as those of vertical turbine pumps. But, the motor of the submersible pump is submerged and is directly connected to and located just beneath the bowl assembly. Water enters through an intake screen between the motor (at lower level) and the bowl assembly (at higher level), passes through various stages, and is discharged directly through the pump column to the surface (6).

The motor of a submersible pump is directly coupled to impellers and is easily cooled because of complete submergence. Ground surface noise is also eliminated. The pump can be mounted in casings which are not entirely straight. The pump house is also not necessary. There are, however, electrical problems associated with submerged cables. These pumps cannot tolerate sand pumping and work less efficiently. The motor is less accessible for repairs and cannot tolerate voltage fluctuations.

Jet Pumps

The jet pump is a combination of a centrifugal pump and a nozzle-venturi arrangement as shown in Fig. 4.15. The nozzle causes increased velocity and reduced pressure at point A. The lowered pressure at A draws additional water from the intake pipe and this water is added to the total volume of water flowing beyond A. The venturi tube helps in the recovery of pressure at B with minimum loss of head. Compared to centrifugal pumps, jet pumps are inefficient but have some advantageous features too. These are adaptable to small wells down to a 5 cm inside diameter. All moving parts of the jet pump are accessible at the ground surface. Their design is simple and results in low equipment and maintenance costs.

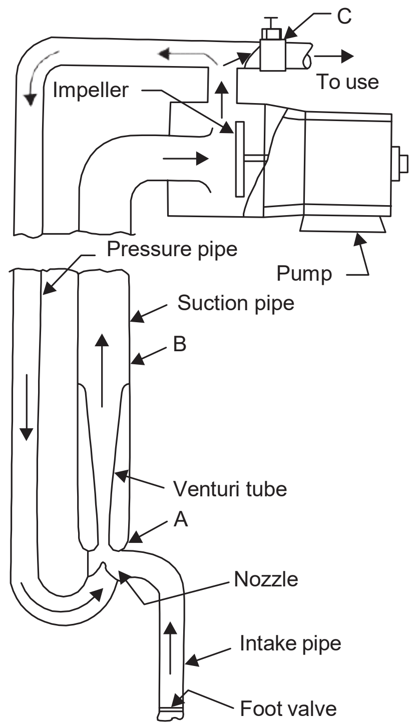


Fig. 4.15 Jet pump

Air-Lift Pumps

Water can also be lifted inside a well by releasing compressed air into an air discharge pipe (air line) lowered into the well. Because of the reduced specific gravity, aerated water is lifted to the ground surface. Air-lift pumping is inefficient and requires cumbersome and expensive equipment and is, therefore, rarely used as a permanent pumping system.

The main factors which must be considered while selecting a pump for water well are the anticipated pumping conditions, specific installation and maintenance conditions, and the basic pump characteristics. As in well construction, the initial cost of a pump and its installation are relatively less important than the performance, reliability, and operating costs during the life span of the pumping equipment.

3.4 Multi-purposes Projects

PLANNING OF WATER RESOURCE PROJECTS

GENERAL

The main aim of all water resource development is to improve the economic and environmental conditions for human living. A water resource project may serve one or more purposes and, accordingly, can be either single-purpose or multipurpose. In most cases, a project would be of a dual or multipurpose type. As such, the entire project needs to be investigated as a unit before the design requirement of a single component, such as a dam, can be finalised. A water resource project may serve one or more of the following purposes:

Irrigation,

Power development,

Flood control,

Industrial water supply,

Domestic and municipal water supply,

Recreation,

Fish and wild life preservation and promotion, and

Navigation.

In almost every water resource project, dam and reservoir are key components of the project. Dams impound water, divert water from a stream, or raise the water level. In exceptionalcases, dams may be constructed to impound water-borne sediments and water having a damaging chemical quality. Dams contribute immensely in reducing poverty and impacts of floods and droughts besides rejuvenating rivers in dry season. Dams also enable recharge of ground water and growth of more biomass. A reservoir is a fresh-water body created or enlargedby the building of dams, barriers or excavations.

It is seldom that a water resource project consists of only a dam and reservoir facility. In a flood control project, levees and other channel control works, besides the dam and reservoir, are usually desirable. Water resource projects for power development and water supply (for irrigation, domestic, municipal, and industrial purposes) have a combination of project components to accomplish the desired objectives. Therefore, dams must be planned, designed, and constructed to operate efficiently and harmoniously with other components of the project to achieve maximum benefits at minimum cost. The economic, environmental and social feasibilities, and justification of dam must be examined in combination with those of other project components, and the total project must be evaluated and judged for its feasibility. If the evaluation of a project proposal does not show justification for its construction, it may be dropped or, alternatively, revised and updated with possible justification at a later time. A water resource project should be planned bearing in mind probable physical, economic, andenvironmental effects.

PHYSICAL FACTORS

Except for flood control projects, availability of sufficient water is essential for all types of water resource projects. In flood control projects, the sudden excess of water is the problem. The source of water is the surface runoff resulting from weather phenomena which are understood only in a general way. Weather conditions can be predicted only as seasonal probabilities. Weather predictions for shorter periods (a few hours or days) can, however, be made with more reliability. Historical measurements of stream flows and rainfall are consideredthe best available means for forecasting stream flow supplies for water resource projects.

At sites where no measurements or only a few measurements have been made, reliable correlation methods are used to estimate streamflow statistics. There is always some risk involved in building a project either too large or too small at sites of meagre stream flow measurements. In such situations, alternatives of staged development or other means of adjusting the project size and scope may have to be considered.

A flood occurring once in 100 years or less may cause enormous damage. Therefore, stream gauging records of 10, 20 or 30 years, though useful to some extent, are inadequate for flood control projects and spillway design for large dams. Besides, actual measurement of peak flood flows is difficult even if the stream is being gauged. Some other methods of estimating the magnitude of peak floods are invariably used for the planning of such works. Computation of the stream flow based on high water marks and flood channel dimensions is one such method. Alternatively, stream flow (or runoff) estimation can be based on actual measurements of amount and duration of high rainfall at rain gauge stations in the catchment area upstream of the dam site. The latter method considers factors such as principles of precipitation as affected by stream characteristics in the region, and the catchment characteristics (location, shape, vegetative cover, and geological structure). Extremely large floods are also extremely infrequent floods. Hence, the planner's judgement is crucial in deciding the size of the flood to be controlled by the project.

Two main factors which determine the site of a water resource project are the areas needing water and the location where water supply is available for development. For economic reasons, the water source must be near the place of use so as to save on cost of conveyance. Also, the source should be at higher elevation than the service area to avoid pumping. In case of projects where water is stored only for the purpose of flood control, there is no conveyance cost involved.

One can build a dam almost anywhere if one spends enough money. But, there is obvious advantage in having a dam site in a narrow section of a stream channel where sufficiently strong and impervious foundation (rock or consolidated material) is available. The abutments must be of sufficient height and be strong and impervious. Further, the dam site should not be located on or very close to an active earthquake fault. The dam site must have suitable site for spillway (a structure which releases surplus water after the reservoir has been filled up to its maximum capacity) which can be made part of the main dam only in case of a concrete dam. A dam requires a very large quantity of construction material (cement, aggregates, impervious and pervious soils, rocks, etc.) which should be available within economical hauling distance of the dam site. An easily accessible site is preferred as it involves least expenditure on communication works required for the transport of construction machinery, power house equipment construction material, and so on, to the dam site. The value of the land and property which would be submerged by the proposed reservoir should be less than the expected benefits from the project.

The area upstream of the dam site would constitute the reservoir component of the project. For economy in dam height, a reservoir site should be wide and on a mildly sloping stream in order to have a long and wide reservoir in proportion to the height of the dam. The reservoir must not be sited on excessively leaky formations. The site with the possibility of landslides, rock-slides or rock falls into the reservoir area (which reduce the storage capacity of the reservoir) must be avoided. The site should not be, as far as possible, on valuable land being used for some other purposes, such as agriculture, forestry, communication, and habitation by people. Sites with mineral deposits in and around the reservoir area should also be avoided. As far as possible, a reservoir should not be provided on a stream carrying large sediment loads which would eventually get deposited in the reservoir, thereby reducing its useful storage capacity. However, all streams carry some amount of sediment. Hence, part of the total reservoir storage is reserved for the accumulation of sediment which is likely to enter the reservoir during its intended economic life. Possibilities of constructing sedimentation basins a short distance upstream of the reservoir and/or providing catchment protection and management against sediment erosion must also be explored.

ECONOMIC CONSIDERATIONS

The cost of a water resource project includes capital investment for constructing the project facilities and the annual or recurring expenditure for operation and maintenance (including replacement) of the project. The capital cost includes the costs of planning, investigations, designs, and construction besides the cost of acquiring rights to the use of water, litigations, and rehabilitation of the affected people. The capital cost also includes the interest on the money invested during construction and up to the start of the project. The benefits likely to be received from a water resource project are widely distributed. As such, the investments on the project cannot be compared with the benefits in terms of monetary units. However, the benefits are expressed, as far as possible, in terms of monetary units and the investment and operational costs are thus compared with the benefits.

It is difficult to quantify some types of project benefits. For example, in an irrigation project, the benefits extend beyond the farmer through a chain of related activities to the people of the area. Social benefits (such as protection against loss of life by floods), recreational benefits, etc. are also difficult to estimate in monetary terms. However, benefits of municipal and industrial water services and hydroelectric power generation can be easily estimated by working out the cost of producing the same results by another reasonable alternative arrangement or by determining the market value of the product. Benefits from a flood control project can be estimated by working out the reduction in flood damages in agricultural, residential, commercial, industrial, and such other activities. The value of the land protected from floods increases and this fact should also be included in the benefits of a flood control project. Other possible benefits from a water resource project may be in the from of a fishery enhancement, water quality improvement (in downstream flows from storage releases during dry seasons), and navigation improvement on large rivers (due to storage releases during low flow seasons). Construction of a water resource project provides employment to people of the locality and is vital in areas of persistent unemployment.

Because of uncertainties involved in the estimation of project benefits, the computed benefit-cost ratio is generally not considered as the sole criterion for determining the economic viability of a project. Nevertheless, such computations do provide a logical basis for arriving at meaningful decisions on the size of the project, inclusion and exclusion of different project functions, the priority of the project, and so on. Other considerations such as social needs, repayment potential, and environmental aspects are also examined in determining the worth of a proposal for water resource development.

ENVIRONMENTAL EFFECTS

A well-planned water resource project should be desirable from economic, social as well as environmental considerations. It should, however, be noted that some of the project components, notably dams and reservoirs, cause adverse environmental effects in the regions of their direct influence. While trying to achieve major project objectives of a water resource project, the planner must examine alternative plans of dams and reservoirs to minimise adverse environmental effects.

Environment is best defined as all external conditions which affect the existence of all living beings. Different living beings affect one another, and the environmental requirementsof different living beings are interrelated.

Besides, it is generally not possible to evaluate environmental effects in economic terms. In case of pollution of water and air, however, it is possible to estimate economic loss to some degree. In addition, it is difficult to assign a degree of importance to various environment conditions likely to be judged differently by different persons depending upon their own viewpoint. For example, the people of a hilly region will have a different viewpoint regarding the siting of a dam from those living in the plains where land is inundated during floods and wells go dry during drought.

The environmental effects which directly affect the livelihood and well-being of people are of prime concern. Other environmental effects on various other living beings are also of concern to man but only to the extent to which the existence of the living beings is important to man's living conditions. The beneficial environmental effects include land use improvements by irrigation, flood protection, improved water supplies for domestic and municipal uses, power supplies without consumption of fuel, water quality improvement, fishery improvement, recreational improvement, and health improvement. Various adverse environmental effects may be caused due to construction and operation of dams and reservoirs. Some of these can be mitigated by taking suitable steps while others are unavoidable. These have been tabulated in Table 14.1. The environmental check-list (Table 1.14) provides a comprehensive guide to the areas of environmental concern which should be considered in the planning, design, operation, and management of water resource projects.

Table 14.1 Adverse effects of dams and reservoirs on environment (1)

Potential adverse effect	Mitigation method or effect	Probable degree or importance of adverse effect
Land use for reservoir		
Loss of fish and aquatic habitat	Changes of species	New species may be less desirable than original species
Loss of wildlife habitat	Improve other areas for habitat	Full mitigation probably not possible
Loss of future access to mineral deposits	None	Is of importance only if mineral deposits exist
Loss of mountain valley areas	None	Important only in extremely mountainous areas
Inundation of historical or archaeological sites	Possibly by a museum	Varies with each individual site
Inundation of exceptional geological formations	Usually not possible	Varies with each individual site
Alteration of downstream flows		

Reduction of fish and aquatic habitat	Maintain regulated flows	Full mitigation possible, but frequently not acceptable because of large sacrifice of project accomplishments
Reduction of stream flushing flows	Release occasional flushing flows	Mitigation method not proven to be worthwhile; Degree of environmental effect depends upon specific stream situation
Changes of water quality	Selective level reservoir outlets; water aeration, if needed	Somewhat limited experience with selective level outlets indicates good prospects of full mitigation
Interference with fish and wildlife migrations		
Blocking anadromous fish runs	Fish hatcheries	Usually capable of full mitigation
Blocking animal migration routes	None practical	Importance depends upon the specific site
Landscape appearance		
Excavation and waste disposal sites	Project expenditures required to landscape sites	Satisfactory mitigation usually possible without excessive expenditure
Reservoir banks below maximum waterline	Minor areas may be developed for beaches	Degree of impact depends upon the specific reservoir site
Abandoned construction facilities	Construction clean-up	Full mitigation possible; important only if not done
Erosion scars from construction roads	Principally by care of drainage	Adverse effects can be reduced but not entirely eliminated within reasonable cost
Reservoir clearing waste disposal	Controlled burning; marketing maximum amounts of wood products	Temporary effect, usually minor, but not entirely avoidable

SELECTION OF A PROJECT PLAN

Planning may be defined as the systematic consideration of a project from the original statement of purpose through the evaluation of alternatives to the final decision on a course of action. Planning of water resource project begins with some definite idea about its main purpose. It is usually economical to have a multipurpose rather than a single-purpose project. From economic considerations, the best project plan is the one for which the ratio of combined project benefits and the total project cost is maximum. The time required to construct a dam and then to first fill the reservoir before the start of the project operation is usually very large (several years) and, hence, the interest on the investment up to the start of the project operation should also be added to the investment costs. The cost of a dam and other major project features and also the benefits for at least three different sizes (the smallest, the largest and an intermediate) of the project are worked out. Using these computations, size-benefit and size-cost curves for different possible functions are prepared. A proper analysis of all this information would yield the size and functions of the project which would result in maximum benefit-cost ratio.

Generally, the needs for water services, power, and flood control in any given region continue to grow due to the increasing population. Therefore, it appears to be uneconomical to build large and costly projects far in advance of their needs. As such, physical and design conditions permitting, a project can also be constructed in stages. Because of the growing concern for environmental conditions, it is essential to take into account the environmental effects of alternative plans. Usually, there is an improvement in the environmental conditions due to the availability of water service and flood protection facilities. However, there are some adverse environmental effects of water resource projects which affect (i) scenic beauty, and (ii) wildlife (both land and aquatic species). These effects, however, cannot be measured. A planner can, therefore, only select an alternative with more favourable or less unfavourable effects.

In making a choice of suitable alternatives, some kind of compromise is always made. These compromises may be in the form of fixing stream flows, acquisition of land to be used as wildlife tracts, siting project features to the advantage of scenic views, and providing access to areas having enjoyment potential. The following method (1) is suggested for this purpose.

Alternative plans of the proposed water resource project, having different amounts of environmental impact but accomplishing other objectives of the project, are prepared. The first step would be to make an inventory of the existing conditions of various important environmental qualities of the water resource system under consideration. These environmental factors may be ranked in order of their importance. The second step in the preparation of alternative plans would be to estimate the future environmental conditions without the project development. These conditions may be the same as the existing ones, or may be degraded or improved. The third step would be to prepare an ''optimum economic water resource project plan alternative'' without considering environmental impacts except those which are positively controlling environmental impacts. Similarly, an ''optimum environmental water resource project plan alternative'' would be prepared wherein an attempt would be made to minimise all adverse environmental impacts and still achieve some of the project objectives. If the second alternative results in significantly reduced accomplishments of the project objectives or greatly increased cost compared to that of the first alternative, the second alternative is discarded in favour of a third alternative plan. The third alternative plan would be so prepared that it would reflect a compromise between the two extreme alternatives and seek to avoid or minimise the important adverse environmental impacts while accomplishing all or most of the project objectives of the first alternative.

The role of the planner of a water resource project is to select the best of all possible alternatives. Various methods of optimization, collectively called systems analysis, are, therefore, obvious tools for this purpose. Because of large number of constraints involved, one has to often make several simplifying assumptions in order to obtain the best possible alternative. Besides, deficiencies of the input data will make determination of the true optimum a difficult task. Nevertheless, systems analysis is still the best method of determining the best possible alternative out of several feasible alternatives.

INVESTIGATIONS

The basic data, usually required for planning of dams and reservoirs, can be grouped in the following categories (1):

Hydrologic data: Stream flows, flood flows, evaporation, sedimentation, water qual- ity, water rights, and tail-water curves.

Geological data: Reservoir sites, dam sites, and construction materials.

Topographic surveys: Catchments, reservoir sites, dam sites, and borrow areas.

Legal data: Water rights.

Reservoir site cost data: Land acquisition, clearing, and relocations.

Environmental factors: Fish and wildlife, recreation, scenic, historical, and archaeo-logical.

Economic data: Economic base for area benefited, crop data, land classification, andmarket data for various purposes.

The desirable quality of these data would depend on the level of investigations. Investigations for a water resource project are generally carried out in three separate steps (or levels or stages): reconnaissance (or preliminary), feasibility, and pre-construction.

Reconnaissance (or Preliminary) Investigations

The main purpose of such investigations is to screen out the poorer alternatives and to decide the types and amounts of more expensive and time-consuming data (such as stream flow records, topographic mapping, and so on) which need to be collected for making feasibility investigations of the remaining selectable alternatives. A reconnaissance survey will identify the scope of a project plan with respect to its geographical location, project functions, approximate size of its various components, likely problem areas, and time and cost of conducting feasibility investigations.

A complete reconnaissance investigation is, in fact, a preliminary version of a feasibility investigation carried out in a rather short time with less accuracy. It considers all the physical, engineering, economic, environmental, and social aspects related to the project. It is usually conducted with the available data. Collection of some new data, if considered necessary for reconnaissance, is made by preliminary surveys. These may include a simple cross-section (instead of detailed topography) of a stream at dam site, surface investigations of geological conditions at dam site, subsurface explorations for foundation quality at dam site, quality and quantity of available construction materials, and so forth. Preliminary designs are made by using short-cut mehods (using curves, tables, and previous experiences). Cost and benefits of the project are also estimated.

Based on the results of preliminary investigations of alternative project plans, a selection of feasible project plans is made for subsequent feasibility investigations.

Feasibility Investigation

The aim of the feasibility investigation is to ascertain the soundness and justification, or lack of these, of different alternative plans chosen after carrying out preliminary investigations. The analyses need to be of high accuracy and dependability so that the reliability of results, on the basis of which the final selection of the project plan is made, may not be questioned. It should, however, be noted that the feasibility investigation does not mean the end of the project planning. Some minor changes are always required to be made for various reasons during final designs before construction, during construction, and even during project operation.

The first step in the feasibility investigation is to collect or update the basic data of different types. The accuracy and reliability levels of these data must be consistent with the degree of accuracy required for feasibility justifications. The basic data for dams and reservoirs include topographic surveys of sites, information on stream flow and design flood, land costs, reservoir clearing costs, communication facilities, climatic conditions affecting construction, fishery and wildlife to be preserved, construction material, foundation conditions of dam site and reservoir area, availability of trained manpower, and important environmental and other considerations. The facilities and appurtenances necessary for the functioning of the project must be specified, and considered while making feasibility cost estimates.

On the basis of the feasibility investigation, a provisional selection of the site and size of the project is made. Besides, the functions of the project are also decided. The final report prepared on the basis of the feasibility investigations is submitted to the approving and funding authorities of the project.

Pre-construction Investigations

The final adoption of provisionally selected project site and its size and functions begins after the project has been approved and funded for construction. It is essential that final designs consider any new information which might have been obtained or received during the time interval between feasibility investigation and the final design. For example, an extreme low runoff season or a flood of large magnitude might have occurred during this intervening period and this may necessitate changes in the estimates of the critical dry year project water supplies or of the frequency of occurrence of a flood of given magnitude.

For pre-construction planning, more detailed and accurate topographic maps and additional geological investigations are usually necessary to reduce uncertainties about foundation conditions and construction material. This is also true of other basic data needed for planning of dams and reservoirs.

CHOICE OF DAMS

Most of the dams can be grouped into one of the following two categories:

Embankment dams, and

Concrete dams.

Embankment dams include earth-fill dams and rock-fill dams. Concrete dams include gravity dams, arch dams and buttress dams. Preliminary designs and estimates will usually be required for different types of dams before one can decide the suitability or otherwise of one type of dam in comparison to other types. The cost of construction is the most important factor to be considered while making the final selection of the type of dam. Besides, the characteristics of each type of dam, as related to the physical features of the site and its adaptation to the purposes of the dam, as well as safety, and other relevant limitations are also to be considered for selecting the best type of dam for a particular site. The following are the important factorswhich affect the choice of the type of dam:

Topography,

Geology and foundation conditions,

Material available, and

Size and location of spillway.

Topography of the site dictates the first choice of the type of dam. A concrete dam would be the obvious choice for narrow stream flowing between high and rocky abutments (i.e., deep gorges). Broad valleys in plains would suggest an embankment dam with a separate spillway.

Geological and hydrogeological characteristics of the strata which are to carry the weight of the dam determine the foundation conditions. Any type of dam can be constructed on solid rock foundations. Well-compacted gravel foundations are suitable for concrete gravity dams of small height, earth-fill, and rock-fill dams. However, effective cutoffs are required to check the foundation seepage. Silt or fine sand foundations can support concrete dams of small height and earth-fill dams. Problems of settlement, piping, and the foundation seepage are associated with this type of foundation. Non-uniform foundations containing different types of strata will usually require special treatment before any type of dam is constructed on such foundations.

If the construction materials to be used in large quantity for the construction of the dam are available in sufficient quantity within a reasonable distance from the site, the cost of the dam will be considerably reduced due to saving on transportation. If suitable soils for the construction of an earth-fill dam are locally available in nearby borrowpits, choice of an earth- fill dam would be the most economical. The availability of sand and gravel (for concrete) nearthe dam site would reduce the cost of a concrete dam.

Spillway is a major part of any dam and its size, type, and the natural restrictions in its location will affect the selection of the type of dam. Spillway requirements are decided by the runoff and streamflow characteristics. As such, spillway on dams across streams of large flood potential can become the dominant part of the dam and put the selection of the type of dam to a secondary position. For large spillways, it may be desirable to combine the spillway and dam into one structure. This is possible only in concrete dams. Embankment dams are based on more conservative design assumptions and, hence, spillway is generally not constructed as part of the embankment. On the other hand, excavated material from a separate spillway can be advantageously used for the construction of an embankment dam.

PLANNING OF RESERVOIRS

One major consideration in the development of any surface water resource project is the structural stability of the reservoir which should be capable of containing safely the projected volumes of water for use throughout its life time. The main factors to be considered for this are as follows (1):

Rim stability,
Water-holding capability,
Loss of reservoir water,
Bank storage,
Seismicity, and
Sedimentation.

Rim stability and water-holding capability are interrelated. Rim failure can be caused due to either the sliding or the erosion of a segment of the reservoir rim. Seepage of water is mainly responsible for such failures. Major slides into a reservoir would, obviously, reduce reservoir capacity considerably. Similarly, snow avalanches and masses of ice falling from hanging glaciers can cause serious problems. Besides reducing the capacity of the reservoir, a rapidly moving slide may also generate waves. A dam may be overtopped due to the resulting wave action or rise of the water surface on account of a major slide into the reservoir. If the reservoir site is likely to be affected by the slides and cannot be abandoned, some restraining steps in reservoir operation should be taken to avoid serious failure. These steps could be in the form of limiting the filling and drawdown rates or imposing the maximum allowable water surface at a level lower than the maximum normal water surface. Alternatively, installation of drains to relieve water pressure along likely slip surfaces, some form of impervious lining, and pinning the unstable mass of its parent formation by rock bolting can be resorted to for preventing slides. Stabilisation of the unstable mass can also be achieved by strengthening or replacing weak material. Grouting is the most common remedy for strengthening such weak masses. It may be desirable to plan the steps to be taken to mitigate the effects of potential slide after it has occurred in spite of all preventive steps.

Reservoir water loss either to the atmosphere or to the ground can be a controlling factor in the selection of a site for a conservation reservoir. For a flood control reservoir, water loss is of concern only if it relates to the safety of the project. The lining of the surface through which seepage is expected is one of the preventive measures to reduce the reservoir water loss to the ground. At times, a blanket of impervious material extending from the heel of the dam is required. This too serves to control the seepage from the reservoir.

Loss of reservoir water to the atmosphere occurs due to direct evaporation from the reservoir surface. The evaporation losses are affected by the climate of the region, shape of the reservoir, wind conditions, humidity, and temperature. From considerations of evaporation, a reservoir site having a small surface area to volume ratio will be better than a saucer-shaped reservoir of equal capacity. Evaporation-retardant chemicals increase the surface tension of water by forming a monomolecular film and thus reduce evaporation.

Bank storage is the water which spreads out from a body of water, filling interstices of the surrounding earth and rock mass. This water is assumed to remain in the surrounding mass and does not continue to move to ultimately join the ground water or surface water as seepage water does. The bank storage is not mitigable. It must, however, be estimated for feasibility investigations and measured during reservoir operation for providing guidelines for reservoir regulation.

It appears that there is some effect of reservoir impoundment on the increased seismic activity of an area in which a large reservoir (having a storage capacity of more than 12×10^8 m^3 behind a dam higher than 90 m) has been constructed (1). However, there have been large reservoirs without increasing the seismic activity of the region. The increased seismic activity is attributed to the changes in the normal effective stresses in the underlying rock because of the increased pore pressure. The transmission of the hydrostatic pressure through discontinuities in the underlying rock can have a triggering effect where a critical state of stress already exists. The relationship between the reservoir impoundment and the earthquake relationship is not fully understood. Hence, it is necessary that every large reservoir site be subjected to detailed geologic, geodetic, and seismic studies for feasibility decision. These observations must be continued during the reservoir operations too in order to better understand the relationship between reservoir impoundment and seismic activity.

The streams bringing water to the reservoir bring sediments too. The sediment gets deposited in the reservoir due to the reduced stream velocity. The capacity of the reservoir is reduced on account of sediment deposition in the reservoir. Usually, a portion of the reservoir storage is reserved for the storage of the sediment. The life of a reservoir is predicted on the basis of the amount of sediment delivered to it, the reservoir size, and its ability to retain the sediment. Sediment deposition at the initial stage may be beneficial in the sense that it may have the effect of a natural blanket resulting in reduced seepage loss. Measures to minimize sediment depostion in reservoirs include catchment protection through a vegetative management programme to prevent soil erosion, silt detention basins at inlets of smaller reservoirs, and low level outlets in dams to provide flushing action for removal of sediment from the reservoir. Of the various measures, the catchment protection is the most effective and also the costliest.

EMBANKMENT DAMS

CHOICE OF DAMS

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Topography,

Geology and foundation conditions,

Material available, and

Size and location of spillway.

Topography of the site dictates the first choice of the type of dam. A concrete dam would be the obvious choice for narrow stream flowing between high and rocky abutments (i.e., deep gorges). Broad valleys in plains would suggest an embankment dam with a separate spillway.

Geological and hydrogeological characteristics of the strata which are to carry the weight of the dam determine the foundation conditions. Any type of dam can be constructed on solid rock foundations. Well-compacted gravel foundations are suitable for concrete gravity dams of small height, earth-fill, and rock-fill dams. However, effective cutoffs are required to check the foundation seepage. Silt or fine sand foundations can support concrete dams of small height and earth-fill dams. Problems of settlement, piping, and the foundation seepage are associated with this type of foundation. Non-uniform foundations containing different types of strata will usually require special treatment before any type of dam is constructed on such foundations.

If the construction materials to be used in large quantity for the construction of the dam are available in sufficient quantity within a reasonable distance from the site, the cost of the dam will be considerably reduced due to saving on transportation. If suitable soils for the construction of an earth-fill dam are locally available in nearby borrowpits, choice of an earth- fill dam would be the most economical. The availability of sand and gravel (for concrete) near the dam site would reduce the cost of a concrete dam.

Spillway is a major part of any dam and its size, type, and the natural restrictions in its location will affect the selection of the type of dam. Spillway requirements are decided by the runoff and streamflow characteristics. As such, spillway on dams across streams of large flood potential can become the dominant part of the dam and put the selection of the type of dam to a secondary position. For large spillways, it may be desirable to combine the spillway and dam into one structure. This is possible only in concrete dams. Embankment dams are based on more conservative design assumptions and, hence, spillway is generally not constructed as part of the embankment. On the other hand, excavated material from a separate spillway can be advantageously used for the construction of an embankment dam.

GENERAL

Embankment dams are water impounding structures composed of natural fragmental materials (such as soil and rock) and consist of discrete particles which maintain their individual identities and have spaces between them. These materials derive strength from their position, internal friction, and mutual attraction of their particles. Unlike cemented materials, these fragmental materials form a relatively flexible structure which can deform slightly to conform to the foundation deflection without causing failure.

Embankment dams have been in existence for many centuries. The earliest forms of these dams were made naturally by landslides and rockfalls which cut off streams and formed natural dams. A 300-m natural dam of this type was created by a landslide which occurred in 1840 on the upper reaches of the Indus river (1). This dam, however, burst just after six months of its formation resulting in great loss of life and property in the valley.

Man-made tanks (or reservoirs) constructed in the early days of civilisation are found in the southern part of India and Sri Lanka. These tanks have been constructed by building earthen embankments. One such earthen embankment 17.6 km long, 21.34 m high, and containing about 13 million cubic metres of earth material was completed in 504 BC (2).

Till around 1925, the methods of design of an embankment dam were based on thumb rules and the heights of such dams rarely exceeded 30 m. The recent developments in soil mechanics have, however, made it possible to design an embankment dam with more confidence. This has resulted in much higher embankment dams such as Beas (116 m) and Ramganga (125 m) dams of India, Goschenenalp dam (156 m) in Switzerland, Oroville dam (224 m) in the USA, Mica Greek dam (235 m) in Canada, and Nurek dam (300 m) in the erstwhile USSR.

Conditions favouring the selection of an embankment dam are as follows (3):

Significant thickness of soil deposits overlying bedrock,

Weak or soft bedrock which would not be able to resist high stresses from a concrete dam,

Abutments of either deep soil deposits or weak rock,

Availability of a suitable location for a spillway, and

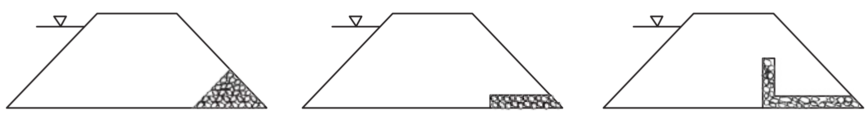
Availability of sufficient and suitable soils from required excavation or nearby bor- row areas.

Embankment dams are mainly of two types:

Earth-fill or earth dams, and

Rock-fill or earth-rock dams.

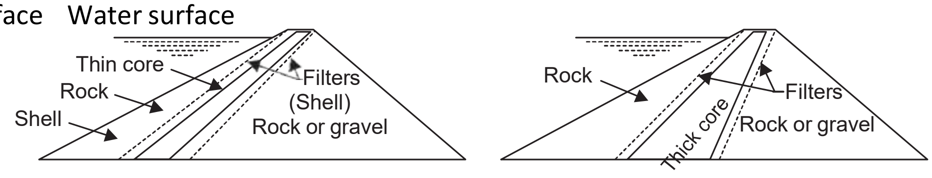
The bulk of the mass in an earth-fill dam consists of soil, while in the rock-fill dam it consists of rock material. The design principles for the two types of embankment dams are similar. Earth dams are further divided into the following types:
Homogeneous earth dam, and
Zoned earth dam.
Homogeneous earth dams are constructed entirely or almost entirely of one type of earth material. A zoned earth dam, however, contains materials of different kinds in different parts of the embankment. A homogeneous earth dam is usually built when only one type of material is economically available and/or the height of the dam is not very large. A homogeneous earth dam of height exceeding about 6 to 8 m should always have some type of drain (Fig. 15.1) constructed of material more pervious than the embankment soil (4). Such drains reduce pore pressures in the downstream portion of the dam and thus increase the stability of the downstream slope. Besides, the drains control the outgoing seepage water in such a manner that it does not carry away embankment soil, i.e., ‘‘piping’’ does not develop. Such a dam is also categorised as homogeneous (sometimes ‘modified homogeneous’) dam (Fig. 15.1). Some of the benefits of a zoned earth dam can be achieved in a homogeneous earth dam by either selective placement of soil or using different construction methods in different parts of the embankment and thus creating zones of different characteristics.



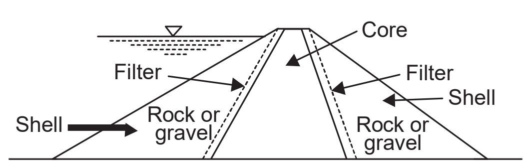
Rock toe or toe drain (b) Horizontal blanket drain (c) Chimney drain

Fig. 15.1 Typical sections of homogeneous earth dam
The most common type of an earth dam usually adopted is the zoned earth dam as it leads to an economic and more stable design of the dam. In a zoned earth dam (Fig. 15.2), there is a central impervious core which is flanked by zones of more pervious material. The pervious zones, also known as shells, enclose, support, and protect the impervious core. The upstream shell provides stability against rapid drawdowns of reservoir while the downstream shell acts as a drain to control the line of seepage and provides stability to the dam during its construction

Water surface Water surface



Thin sloping core dam (b) Thick sloping core dam



(c) Central core dam

Fig. 15.2 Typical sections of zoned earth dam and operation. The central core provides imperviousness to the embankment and reduces the seepage. The maximum width of the impervious core will be governed by stability and seepage criteria and also by the availability of the material. An earth dam with a sufficiently thick impervious core of strong material with pervious outer shells can have relatively steeper embankment slopes limited only by the foundation and embankment characteristics. However, a thin core dam is usually more economical and more easily constructed because of lesser amount of fine-grained soil to be handled. Core widths of 30 to 50% of the water head are usually adequate for any type of soil and any dam height while core widths of 15 to 20% of water head are thin and considered satisfactory, if adequately designed and constructed filter layers are provided (4). Core widths of less than 10% of water head should not be used as far as possible.

The impervious core can be placed either as a vertical core or as an upstream sloping core, each of which has some advantages over the other. A vertical core results in higher pressure on the contact between the core and foundation which, in turn, reduces the possibility of leakage along the contact. Besides, for a given quantity of impervious material, the vertical core will have greater thickness. The main advantage of upstream sloping core is that the main downstream shell can be constructed first and the core placed later – an advantageous feature in areas which have short periods of dry weather suitable for building a core of fine- grained soil. Besides, foundation grouting can be carried out while the downstream embankment is being constructed.

A rock-fill dam (Fig. 15.3) is an embankment which uses large-sized rock pieces to provide stability and an impervious membrane to provide watertightness (5). Materials used for the membrane are earth, concrete, steel, asphalt, and wood. The impervious membrane can be placed either on the upstream face of the dam or as a core inside the embankment. The upstream face of the dam is, however, more suitable for placing the impervious membrane due to the following reasons (4):

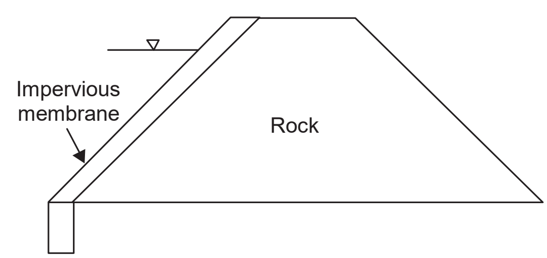


Fig. 15.3 Typical section of a rock-fill dam

The upstream impervious membrane, with a suitable drain behind it, prevents seep- age from entering the embankment. This reduces pore pressures and prevents the embankment mass from being submerged. Both these effects result in greater stabil- ity of the embankment.

The upstream impervious membrane is accessible for inspection and repair.

The upstream impervious membrane also serves a secondary function of wave pro- tection.

The upstream impervious membrane can be built after completion of the embank- ment. This would permit initial settlement of the embankment without affecting the membrane adversely.

DESIGN CONSIDERATIONS

The design of an embankment dam is based on analytical considerations as well as on experience.The main steps in the design of an embankment dam are as follows (4):

A thorough exploration of the foundation and abutments.

Evaluation of the quantities and characteristics of all the embankment construction materials available within a reasonable distance of the dam site.

A study of all the factors which may influence the design.

The selection of trial designs.

Analysis of the safety of the trial designs.

The modification of the designs to satisfy the minimum stability requirements.

The preparation of detailed cost estimates.

The final selection of the design which seems to offer the best combination of economy,safety, and convenience in construction.

The steps (iii) and (v) have been discussed in the following sections.

Factors Influencing the Design of an Embankment Dam

Materials Available for Construction

One of the main advantages of an embankment dam is the availability of construction material free of charge at or near the dam site. Depending upon the type of material available, the designed embankment may either be a homogeneous earth dam (when the soil available is impervious), a zoned earth dam (when both pervious and impervious soils are available) or a rock-fill dam (if rock is available and impervious material is not). The design may also incorporate use of materials from required excavation (for spillway construction) for reasons of economy.

Foundation Characteristics

An embankment dam can be constructed on almost any kind of foundation. Foundation characteristics mainly affect the foundation treatment which, in some cases, may be the most difficult and important part of the design and construction of an embankment dam. Besides, the embankment dimensions would be considerably influenced. For example, a softer foundation would necessitate an embankment with flatter slopes, broader cross-section, a larger freeboard (to mitigate the effects of embankment settlement), considerations for differential settlement cracks, and measures for control of underseepage to avoid the danger of piping.

Climate

It is generally difficult to handle fine-grained soils during the rainy season and control the construction moisture content of the fine-grained soils in arid regions. As such, if the constructionof the embankment has to be carried out during the rainy season, it is advisable to have sloping core embankment. Similarly, in arid regions, one extra year may be required for constructing a small reservoir for storing flood runoff for the purpose of construction of the dam.

Shape and Size of Valleys

A dam site with broad valleys and gently sloping abutments may not affect the design of an embankment. However, narrow valleys and steep abutments may necessitate special design provisions. For example, because of the limited working space in a narrow valley, a simpler design requiring few special construction provisions is preferable. If the construction and maintenance of haul roads on the abutments at different elevations become difficult and costly, one may have to design a rock-fill embankment which can be constructed by dumping rock in high lifts from relatively few haul roads.

River Diversion

If a river diversion scheme is to be implemented by the construction engineer or the contractor, it increases the problem of the designer who must envisage all possible ways of river diversion and make his design adaptable to each of these ways. On major rivers, however, it may be advisable to specify the river diversion scheme and design the embankment accordingly. In a narrow valley, the river is diverted through a tunnel or conduit. In wider valleys, parts of the embankment on the two abutments are constructed while letting the river flow through the central region of the valley. This central part of the embankment is constructed only at the end and is known as the 'closure' section. The construction of the closure section is carried out rapidly to prevent overtopping of the dam and, hence, special design details (viz., providing extra filter drains, different designs for different embankment sections in order to use the material available on the two abutments, and so on) and construction details (such as compactingthe closure section at higher water content, keeping a reserve of borrow material to achieve a rapid construction rate for closure, etc.) are specified. If coffer dams of large volumes are used for diversion purposes, it would be economical to incorporate these into the dam embankment, if possible.

Probable Wave Action

The severity of the wave action and the amount of protection needed for the upstream face of the embankment mainly depends on the wind velocity and the length of the reservoir. The waves drive repeatedly against the embankment and, thus, cause the embankment erosion. A layer of dumped rock riprap is considered the most effective and economical wave protection.

Time Available for Construction

The design of an embankment is dependent on the time available for construction. A shorter construction period, in case of high dams, may result in higher pore pressures requiring relatively flatter slopes. When construction time is limited, it may not be possible to use the material from the required excavation, or it may be that only a part of it can be used. Similarly, underseepage measures would also be affected by the time available for construction. Handling of fine-grained soils requires considerable time and, therefore, it may be desirable to provide a manufactured impervious membrane to save time.

Function of the Reservoir

The function of the reservoir determines the allowable water loss due to seepage through the embankment and foundation. Accordingly, the embankment section may be relatively more impervious (for conservation reservoirs) or relatively more pervious (for flood control reservoirs). In hydroelectric projects, the upstream face of the dam will be subjected to a ''sudden drawdown'' condition which may necessitate the provision of a flatter upstream slope.

Earthquake Activity

In regions of seismic activity, the designer may have to adopt more conservative design features such as better filters, downstream drains of larger capacity, thicker cores of more piping resistantmaterials, flatter side slopes, longer construction time, and so on.

General Design Criteria for Embankment Dams

The major causes of failure of an embankment dam are overtopping, piping, and earth slides in a portion of the embankment and its foundation (due to insufficient shear strength). Ofthese, overtopping is the most common cause of complete and catastrophic failure of an embankment dam. The design of an embankment dam must meet the following safety requirements:

There is no danger of overtopping. For this purpose, spillway of adequate capacity and sufficient freeboard must be provided.

The seepage line is well within the downstream face so that horizontal piping may not occur.

The upstream and downstream slopes are flat enough to be stable with the materials used for the construction of embankment for all conditions during construction, op- eration, and sudden drawdown.

The shear stress induced in the foundation is less than the shear strength of the foundation material. For this purpose, the embankment slopes should be sufficiently flat.

The upstream and downstream faces are properly protected against wave action and the action of rain water, respectively.

There should not be any possibility of free passage of water through the embank- ment.

Foundation seepage should not result in piping at the downstream toe of the dam.

The top of the dam must be high enough to allow for the settlement of the dam and its foundation.

The foundations, abutments, and embankment must be stable for all conditions of operation (steady seepage and sudden drawdown) and construction.

Freeboard

All embankment dams must have sufficient extra vertical distance between the crest of the embankment dam and the still water surface in the reservoir. This distance is termed the freeboard (7) and must be such that the effects of wave action, wave run-up, wind set-up, earthquake, and settlement of embankment and foundation do not result in overtopping of the dam. Normal freeboard is measured with respect to the full reservoir level while the minimum freeboard is measured with respect to the maximum water level in the reservoir (7). An adequate freeboard is the best guarantee against the failure of an embankment dam due to overtopping.

Wave action for freeboard computations is best represented by the wave height and wave length both of which depend on fetch and wind velocity. Fetch is defined as the maximum straight line distance over open water on which the wind blows (7). Effective fetch is weighted average fetch of water spread, covered by 45° angle on either side of trial fetch, assuming the wind to be completely non-effective beyond this area. The effective fetch is calculated (7) by drawing fifteen radials on the reservoir contour map at an interval of 6° from a selected point located on the periphery at which fetch is required to be determined (Fig. 15.4). The central radial is drawn in the direction of wind. It should be noted that after the construction of reservoir, the funnelling action of the valley may direct wind towards the dam. As such, the wind direction may preferably be assumed along the maximum fetch line. Each radial runs the full length of the water surface at a given pool elevation. Effective fetch f_e is calculated from

$$f_e = \frac{\sum R_i \cos \alpha_i}{\sum \cos \alpha_i} \qquad (15.1)$$

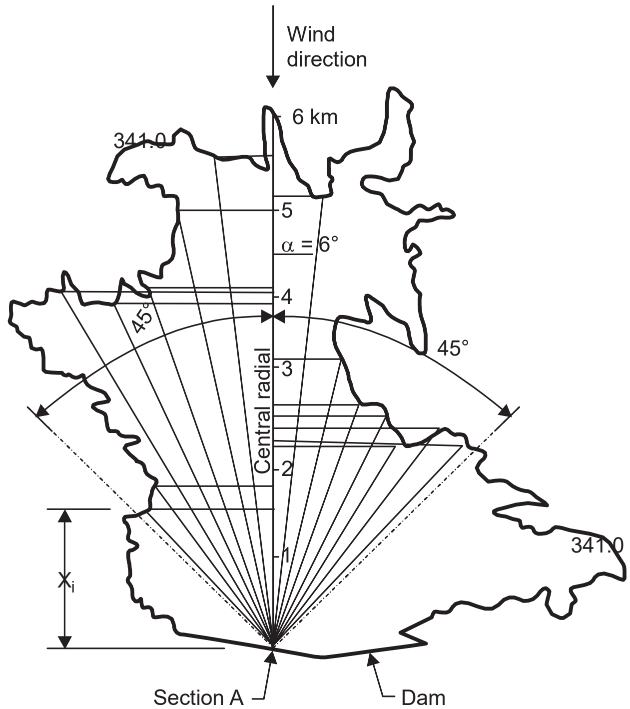


Fig. 15.4 Radials for computations of effective fetch

Here, R_i is the effective length of the i^{th} radial and α_i is the angle between the i^{th} radial and the central radial. Such values of effective fetches are calculated for 2 or 3 different trial fetches and the maximum value of these effective fetches is used for further computations of wave height H_s and the wave period T_s which are, respectively, given as (7)

$$\frac{gH_s}{V^2} = 0.0026 \left(\frac{gf_e}{V^2}\right)^{0.47} \quad (15.2)$$

and

$$\frac{gT_s}{V} = 0.45 \left(\frac{gf_e}{V^2}\right)^{0.28} \quad (15.3)$$

The wave length, L_s (in metres) is obtained from

$$L_s = 1.56\, T_s^2 \quad (15.4)$$

Here, H_s and T_s are the wave height (in metres) and wave period (in seconds) of a significant wave, g the acceleration due to gravity (m/s^2), V the wind velocity (m/s) over the water surface, and f_e is the effective fetch (in metres).

The wind velocities over the water surface are higher than the wind velocities over the land surface and the difference between the two depends upon the fetch and the surrounding terrain conditions. The ratio of the wind velocity over the water surface to the wind velocity over the land surface is given in Table 15.1. For computation of minimum freeboard, the wind velocity is taken as half to two-thirds the wind velocity adopted for calculating normal freeboard (7). For calculation of the normal and minimum freeboard, the wave length and design wave height (H_0) are taken as L_s and 1.67 H_s, respectively (7).

Table 15.1 Ratio of the wind velocity over water surface to the wind velocity over land surface (7)

Effective fetch (km)	1	2	4	6	8	10 and above
The ratio	1.1	1.16	1.24	1.27	1.3	1.31

Wave run-up is the vertical difference between the maximum elevation attained by wave run-up on a slope and the water elevation on the slope excluding wave action. The wave run-up, R for a smooth surface can be obtained from Fig. 15.5. Wave run-up on a rough surface would be less and, hence, the values of R, obtained from Fig. 15.5, are multiplied by a correction factor obtained from Table 15.2.

Table 15.2 Surface roughness correction factor for wave run-up (7)

Type of pitching	Recommended correction factor
Cement concrete surface Flexible brick pitching Hand-placed riprap laid flat laid with projections Dumped riprap	1.00 0.80 0.75 0.60 0.50

2.8

2.4

2.0

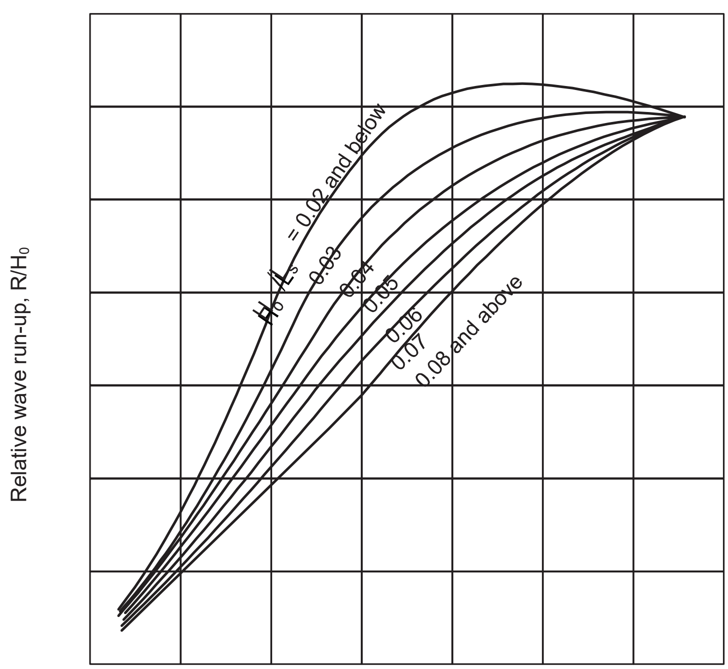
1.6

1.2

0.8

0.4

0



0 0.1 0.2 0.3 0.4 0.5 0.6 0.7

Embankment slope

Fig. 15.5 Relative run-up of waves

If the wave run-up is less than the design wave height H_0, the freeboard is governed by the design wave height H_0 (7).

Wind set-up is the result of piling up of the water on one end of the reservoir on account of the horizontal driving force of the blowing wind. The magnitude of the rise of water surface above the still water surface is called the wind set-up or the wind tide (7).

Consider sections 1 and 2, Fig. 15.6, in a reservoir. Let the distance between these sections be dF. Let the depth at section 1 be D. The wind exerts shear stress τ_w on the water surface (assumed horizontal) as a result of which the depth at section 2 is D + dS. The shear on the bed of the reservoir is, say τ_b. Applying momentum principle to the control volume between sections 1 and 2, one gets

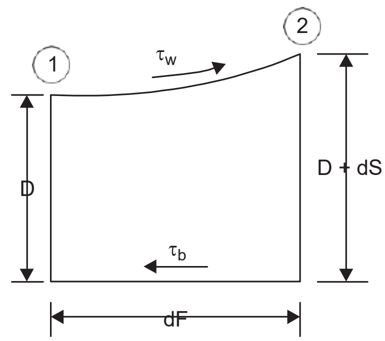


Fig. 15.6 Wind set-up

$$\tau_w\, dF - \tau_b\, dF + \frac{1}{2}\rho g D^2 - \frac{1}{2}\rho g (D + dS)^2 = \rho Q (U_2 - U_1)$$

Here, ρ is the mass density of water, Q the discharge, and U_1 and U_2 are the velocities of flow at sections 1 and 2, respectively. Since the depth in the reservoir is large, the velocities U_1 and U_2 are negligible and, therefore, the shear stress τ_b too is negligible. If one drops the term containing square of dS (as dS is small), the above equation reduces to

$$\tau_w\, dF = \rho g D dS$$

$$\therefore \quad dS = \frac{\tau_w dF}{\rho g D}$$

Since, τ_w is proportional to the square of the wind velocity V, the wind set-up S can be written as

$$S \propto \frac{V^2 F}{D}$$

In practice, the wind set-up, S is calculated from the Zuider Zee formula (7):

$$S = \frac{V^2 F}{62{,}000 D} \qquad (15.5)$$

where, V is the velocity of the wind over the water surface (in km/hr), F the fetch in km, and D is the average depth of water in metres along the maximum fetch line.

The freeboard is thus the sum of the height (or wave run-up) and the wind set-up. In order to account for the uncertain effects of seiches (periodic undulations in the reservoir water surface believed to be on account of earthquake, intermittent wind, varying atmospheric pressures, and irregular inflow and outflow of water), and vertical settlement of the embankement and foundation, an additional margin of safety is always added to the computed freeboard. The freeboard (normal as well as minimum) should not, however, be less than 2 m (7). The elevation of the top of the embankment dam is obtained by adding normal freeboard to the full reservoir level and the minimum freeboard to the maximum water level in the reservoir. Obviously, the higher of the two elevations is to be adopted as the elevation of the top of the dam.

Example 15.1 Compute freeboard and the level of top of the dam for the following data:

Full reservoir level	= 340.00 m
Maximum water level	= 342.20 m
Effective fetch	
for normal freeboard	= 3.66 km
for minimum freeboard	= 4.00 km
Wind velocity over land for normal freeboard	= 150 km/hr
Average depth of reservoir	
for normal freeboard	= 29.0 m
for minimum freeboard	= 31.2 m
Embankment slope	= 2.5 (H) : 1(V)

The upstream face is covered with hand-placed stone pitching.

Solution:

Quantity	For normal freeboard	For minimum freeboard	Remarks
Effective fetch (km)	3.66	4.00	
Wind velocity over land (km/h)	150.00	75.00**	
Wind coefficient	1.226	1.240	From Table 15.1
Wind velocity over water,			
V(km/hr)	183.90	93.00	
V(m/s)	51.083	25.83	
Significant wave height, H_S (m)	2.37	1.20	From Eq. (15.2)
Wave period, T_S (s)	4.88	3.71	From Eq. (15.3)
Wave length, L_S (m)	37.15	21.47	From Eq. (15.4)
Design wave height, H_0 (m)	3.96	2.00	$H_0 = 1.67\ H_S$,
Wave steepness, H_0/L_S	0.1066	0.093	
Relative wave run-up, R/H_0	1.6	1.6	From Fig. 15.5
Wave run-up, R (m)	6.336	3.20	
Surface roughness correction factor for wave run-up	0.75	0.75	From Table 15.2
Corrected wave run-up (m)	4.752	2.4	
Wind set up, S (m)	0.069	0.018	From Eq. (15.5)
Freeboard required (m)	4.752 + 0.069 = 4.821	2.4 + 0.018* = 2.418	
Elevation of top of dam (m)	340.0 + 4.821 = 344.821	342.2 + 2.418 = 344.618	

** Assuming wind velocity for minimum freeboard to be half the wind velocity for normal freeboard (7).
* Wave run-up is higher than design wave height (H_0)
Therefore, adopt elevation of top of the dam as 344.821 or, say, 345.00 m.
ESTIMATION AND CONTROL OF SEEPAGE

Seepage Analysis

The theory of flow through porous media is utilised for the estimation of seepage through an embankment dam and its foundation. As discussed in Sec. 9.3, the governing equation for two-dimensional seepage, as would occur in an embankment dam and its foundation, is the Laplace equation

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} = 0 \quad (15.6)$$

Here, h is the seepage head. Equation (15.6) is valid for homogeneous, isotropic, and incompressible soil which is fully saturated with incompressible water. Equation (15.6) can be solved by graphical, analytical, numerical, or some other suitable methods, such as analogue methods. The graphical solution of Eq. (15.6) involves drawing of flownet which has been discussed in Sec. 9.3.

Most of the seepage problems related to embankment dams can be analysed by drawing flownets for sections with single permeability. For example, if the outer shells of a dam are many times pervious than the core, the analysis of the seepage conditions in the core alone may be adequate for such cases. However, in many seepage problems one has to analyse seepage through sections of different permeabilities. For such conditions, a basic deflection rule mustbe followed in passing from a soil of one permeability to a soil of different permeability. Seepage water needs less energy to flow through a region of relatively higher permeability. Therefore, when water flows from a region of high permeability to one of lower permeability, the flow takes place in such a manner that it remains in the more permeable region for the greatest possible distance. In other words, water seeks the easiest path to travel in order to conserve its energy. Another way of appreciating the seepage behaviour in sections of different permeabilitiesis that, other factors being the same, smaller area of flow cross-section is needed in the higherpermeability region.

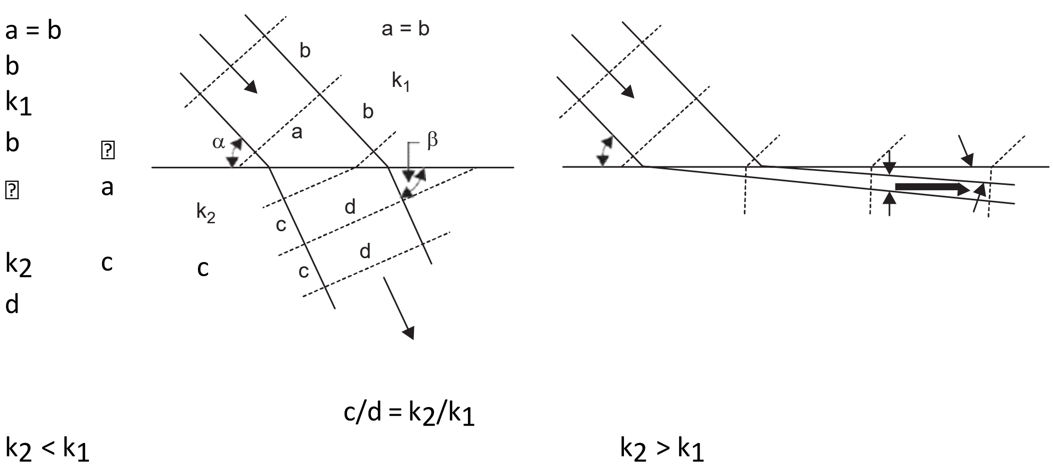


Fig. 15.7 Change in shape of flownet squares on account of regions of different permeability

In seepage through porous medium, the hydraulic gradient is the measure of the rate of energy loss. One would, therefore, expect steep hydraulic gradients in zones of low permeability.

Figure 15.7 shows the deflection of flow lines when they cross boundaries between soils of different permeabilities. The flow lines bend in accordance with the following relationship (8):

$$\frac{\tan \alpha_1}{\tan \alpha_2} = \frac{K_1}{K_2} \qquad (15.7)$$

Also, the areas formed by the intersecting flow and equipotential lines either elongate or shorten according to the following relationship (8):

$$\frac{c}{d} = \frac{K_2}{K_1} \qquad (15.8)$$

While drawing flownets for sections with different permeabilities, one must regularly measure the lengths and widths of the figures to ensure that Eq. (15.8) is satisfied.

At times, the compacted embankments and natural soil deposits are stratified rendering them more permeable in the horizontal direction than in the vertical direction. For this anisotropic condition, the velocity components, Eq. (9.42), for two-dimensional flow will be

$$u = -K_x \frac{\partial h}{\partial x} \quad \text{and} \quad v = -K_y \frac{\partial h}{\partial y} \qquad (15.9)$$

Thus, for two-dimensional flow, the continuity equation, Eq. (9.43), with $\partial w/\partial z = 0$ and combined with Eq. (15.9), yields

$$K_x \frac{\partial^2 h}{\partial x^2} + K_y \frac{\partial^2 h}{\partial y^2} = 0 \qquad (15.10)$$

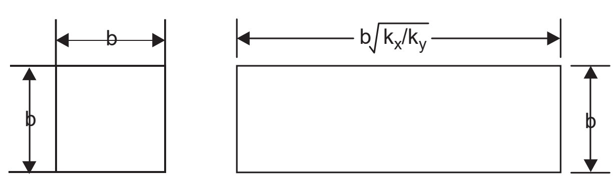
Equation (15.10) can, alternatively, be written as

$$\frac{\partial^2 h}{\partial x_t^2} + \frac{\partial^2 h}{\partial y^2} = 0 \qquad (15.11)$$

where, $x_t = x\sqrt{K_y/K_x}$

Equation (15.11) is the familiar Laplace equation with transformed coordinate system involving x_t and y, and governs the flow in anisotropic seepage condition. To draw a flownet for the anisotropic condition, one needs only to shrink the dimensions of the given cross-section in the direction of greater permeability (8). Having drawn the flownet for the transformed section, it is then reconstructed on the cross-section drawn to the original scale. Obviously, the flownet on the original cross-section would not be composed of squares but of rectangles elongated in the direction of greater permeability.

The effective permeability K can be determined by comparing the discharge Δq through any one figure of the transformed section and the corresponding figure of the original section (Fig. 15.8). If Δh is the drop in head between adjacent equipotential lines, then



(II)

Fig. 15.8 Comparison of flownet figures in the (I) transformed and (II) original section

$$\Delta q = K\frac{\Delta h}{b}b = K_x\frac{\Delta h}{b\sqrt{K_x/K_y}}b$$

$$\therefore \quad \bar{K} = \sqrt{K_x K_y} \qquad (15.12)$$

One can also determine the seepage quantity using Eq. (9.46) which is rewritten as

$$q = Kh\frac{N_f}{N_d} \qquad (15.13)$$

Equation (15.13) is valid for the isotropic condition but can also be used for the anisotropic condition by replacing K with effective permeability $\bar{K}$ [Eq. (15.12)]. It should be noted that the shape factor (i.e., N_f/N_d) remains the same for both the original and transformed sections. The major difficulty in the seepage analysis of an embankment dam is that the topmost streamline, i.e., the seepage line or the phreatic line, is not known. The seepage line is defined as the line above which there is no hydrostatic pressure and below which there is hydrostatic pressure (6). If the embankment is composed of coarse material, the capillary effects are negligible and the seepage line is practically the line of saturation. But, in case of an embankment of fine-grained soil, there is saturation without hydrostatic pressure and also a negligible flow occurs in the capillary fringe above the seepage line. The prediction of the seepage line helps in drawing the flownent and determines the piping potential. In case of a homogeneous earth dam founded on impervious foundation, the seepage line cuts the downstream face above the base of the dam unless, of course, special drainage measures are adopted.

The equipotential lines must intersect the seepage line at equal vertical intervals (Fig. 15.9). This requirement permits graphical determination of the seepage line simultaneously while a flownet is being drawn. Alternatively, one can determine the seepage line using Kozeny's solution according to which the equation of seepage line for an embankment with parabolic upstream face and downstream horizontal drain, shown in Fig. 15.10, is expressed as

$$y^2 - y_0^2 + 2xy_0 = 0 \qquad (15.14)$$

where, $y_0 = 2a_0 = \sqrt{d^2 + h^2} - d \qquad (15.15)$

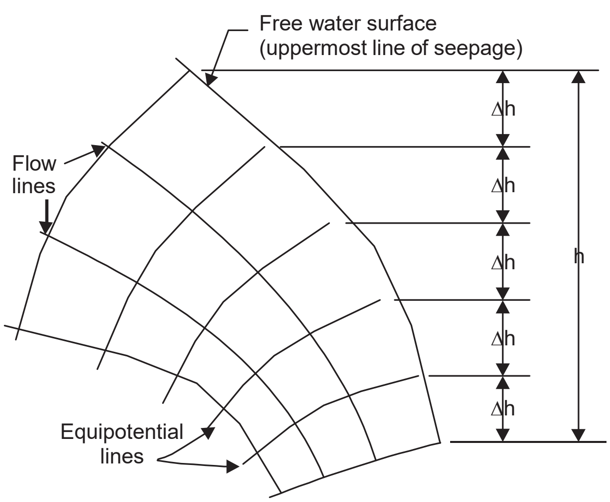


Fig. 15.9 General conditions for line of seepage

Further, the seepage discharge per unit length of embankemnt, q, through the embankment, as per Kozeny's solution, is given as

$$q = Ky_0 = K\left(\sqrt{d^2 + h^2} - d\right) \qquad (15.16)$$

where, K is the coefficient of permeability, and other symbols are as explained in Fig. 15.10.

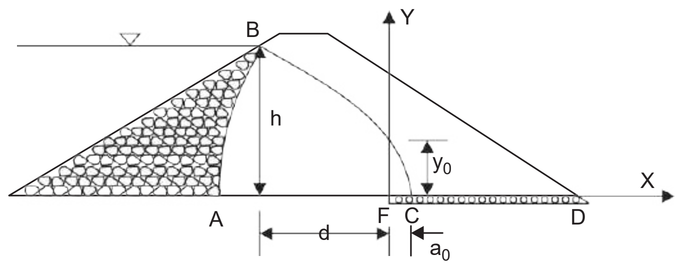


Fig. 15.10 Embankment dam with Kozeny's conditions

It should be noted that the location of seepage line and the point at which it cuts the downstream face (the downstream drain in Fig. 15.10) are dependent only on the cross-section of the dam and are not affected by the coefficient of permeability of the embankment material. Further, actual embankment dams do not have parabolic upstream faces though many of them have downstream horizontal drain. The seepage line follows a parabola, Eq. (15.14), with some departures at the entry and the exit of common types of embankment dams.

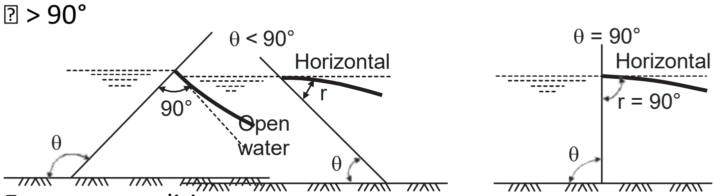
Casagrande's Solution for Common Embankment Dams

Casagrande (9) has described important conditions which must be met by flownets at the points of entry, discharge, and transfer across boundaries between dissimilar soils. These conditions have been given in Fig. 15.11. Casagrande (9) has also extended Kozeny's solution for common embankment dams with usual upstream face and the downstream drain other than the horizontal drain.

Casagrande obtained accurate solutions for different embankment sections by drawing flownet and compared the seepage lines with Kozeny's parabolic seepage line (also called 'base parabola'). He, thus, found that in the central portion, the seepage line coincided with the base parabola. Further, the base parabola meets the water surface at the corrected entry point B which lies on the water surface, and is at a distance of 0.3 times the horizontal projection of the water-covered upstream face, from the junction of the water surface with the upstream face (Fig. 15.12). This means that EB = 0.3 EG. The actual seepage line, however, starts from E and is at right angles to the upstream face which is an equipotential surface. It, then, takes a reverse curvature and meets the base parabola tangentially.

When the downstream drain is other than a horizontal drain (in which case angle of the discharge face, α = 180°), the actual seepage line departs from the base parabola in the exit region also. For α less than 90°, the seepage line meets the discharge face, i.e., the downstream slope, tangentially at C. For 90° < α < 180°, the seepage line becomes vertical at the discharge face. Comparing his graphical solutions with the corresponding base parabola, Casagrande obtained a graphical relation between the ratio $\Delta a/(a + \Delta a)$ and the angle of discharge face α (Fig. 15.13). Here, a is the distance from the focus F, along the discharge face, to the point where the seepage line meets the discharge face while $(a + \Delta a)$ is the corresponding distance for the base parabola. Using this graph, □a and, hence, the corrected exit point C can be determined. The base parabola in the exit region is, therefore, suitably modified so that it meets the discharge face at C.

□ > 90°



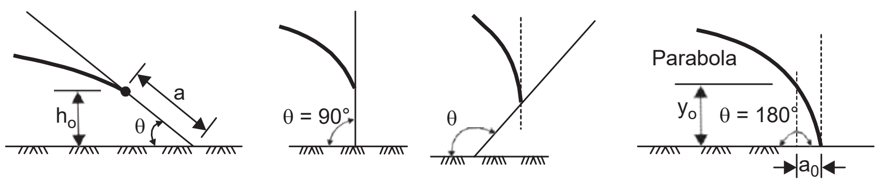
Entrance conditions

For □ □ 90° line of seepage tangent to discharge face

For 90° □ □ □ 180° line of seepage tangent to vertical at point of discharge

Vertical

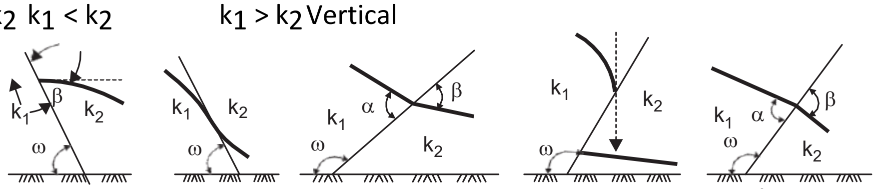
Vertical



Exit (or discharge) conditions

$k_1 > k_2$ $k_1 < k_2$ $k_1 < k_2$ $k_1 > k_2$ Vertical

□□



□ = □ = □

□ = □ = 0

□

Exceptional case

□ = 270° – □ – Discontinuity, usual case

□ = 270° – □ – □

Transfer conditions at boundaries between dissimilar soils

Fig. 15.11 Different conditions for seepage line

G

B E

K

EB = 0.3 EG

Top flow line

C

Base parabola □

F

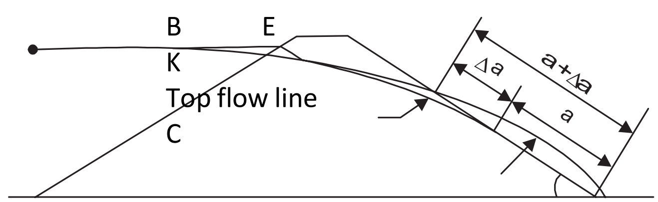
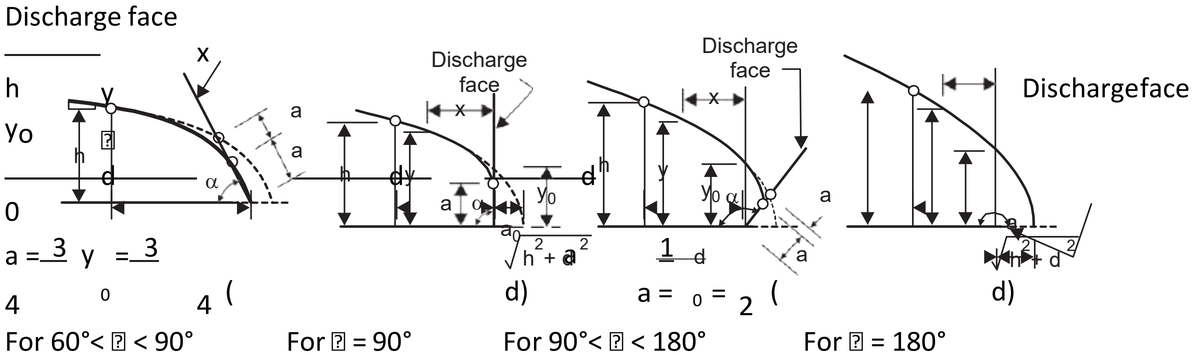


Fig. 15.12 A Casagrande's solution

The seepage quantity can be determined by using Kozeny's equation [Eq.(15.16)] with d being replaced by the horizontal distance between the corrected entry point B and focus F ofthe base parabola. Alternatively, one can draw the flownet and use Eq.(15.13).



Diagrams for determining a + Δa

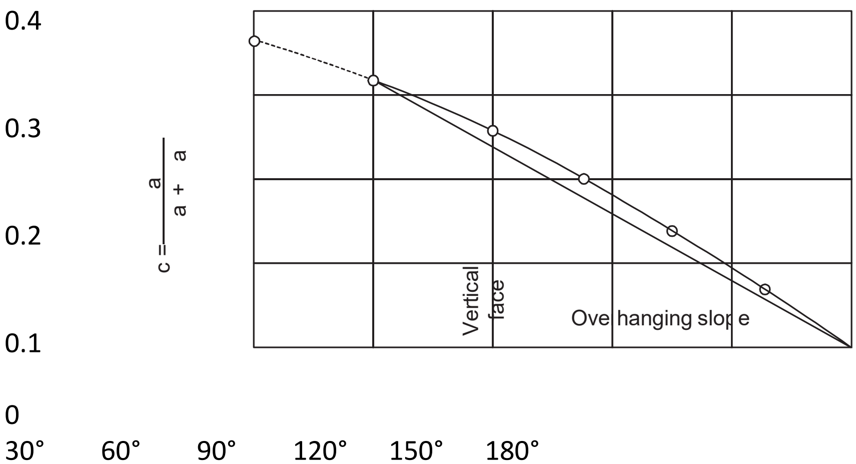


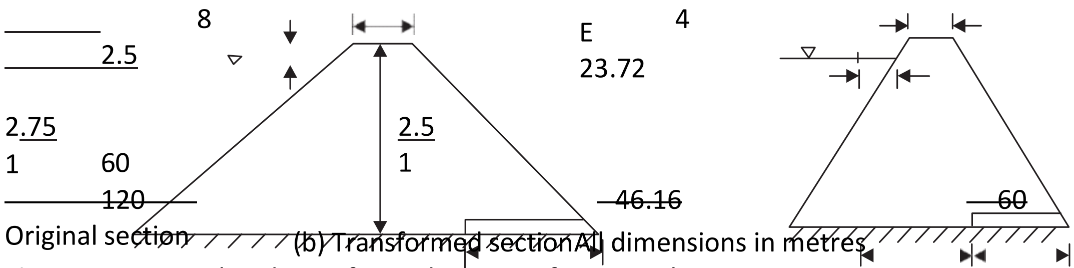
Diagram for determining Δa and a

Fig. 15.13 Correction at the discharge point of base parabola

Example 15.2 Calculate the seepage through an earth dam [Fig. 15.14 (a)] resting on an impervious foundation. The relevant data are as follows:

Height of the dam	= 60.0 m
Upstream slope	= 2.75 : 1 (H : V)
Downstream slope	= 2.50 : 1 (H : V)
Freeboard	= 2.5 m
Crest width	= 8.0 m
Length of drainage blanket	= 120.0 m

Coefficient of permeability of the embankment material in x-direction = 4×10^{-7} m/s
in y-direction = 1×10^{-7} m/s



Original section (b) Transformed section All dimensions in metres

Fig. 15.14 Original and transformed sections for Example 15.2

Solution: Since it is a case of anisotropic permeability, the original section needs to be transformed through the transformation

$$x_t = x\sqrt{K_y/K_x} = x\sqrt{1 \times 10^{-7}/4 \times 10^{-7}} = x/2$$

With respect to the transformed section [Fig. 15.14 (b)]

d = 46.16 m and h = 57.5 m

and $\bar{K} = \sqrt{K_x K_y} = \sqrt{4 \times 10^{-7} \times 1 \times 10^{-7}} = 2 \times 10^{-7}$ m/s

From Eq. (15.16)

$$q = \bar{K}\left[\sqrt{d^2 + h^2} - d\right]$$

$$= 2 \times 10^{-7}\left[\sqrt{46.16^2 + 57.5^2} - 46.16\right]$$

$$= 55.2 \times 10^{-7}\ m^3/s/m$$

Methods of Controlling Seepage through Embankmentand its Foundation

Seepage through an embankment dam and its foundation causes not only the loss of water but, if uncontrolled, also results in piping in the downstream portion of the embankment and foundation. Besides, the stability of slopes is also affected by seepage forces. Piping is the progressive erosion of embankment material due to leaks which develop through the embankment or foundation. Leaks in an embankment are usually caused because of poor construction resulting in insufficiently-compacted or pervious-layered embankment, inferior compaction adjacent to concrete outlet pipes or other structures, and weak bond between the embankment and the foundation or abutments. Piping is caused due to the erosive forces of seepage water tending to move soil particles along with the seepage water. When the forces resisting erosion (viz., cohesion, the interlocking stresses, the weight of soil particles as well as the action of the downstream filters, if present) are less than the erosive forces of the seepage water, the soil particles are washed away and piping begins. Earth slides (or sloughing) are related to piping. Sloughing begins when a small amount of material at the downstream toe has eroded and caused a small slump (or slide) leaving behind steeper slope which eventually gets saturated due to seepage water and slumps again. This may continue and cause complete failure of the embankment.

In order to check these effects of seepage, some measures must be taken to control the seepage in embankment dams so that the seepage line is well within the downstream face and the seepage water is suitably collected and disposed of. The methods of seepage control for embankment dams can be grouped into two broad categories:

Methods which prevent or reduce the seepage, such as complete vertical barriers (viz., rolled-earth cutoffs, steel sheet piles, and concrete walls), grout curtains, up- stream impervious blanket, and thin sloping membrane.

Methods which control the seepage water that has entered, such as embankment zoning, horizontal blanket drains, chimney drain, partially penetrating toe drains, and relief wells.

Usually, a combination of these methods is used.

Rolled-Earth Cutoff

Rolled-earth cutoff [Fig. 15.15 (a and b)] provides an effective barrier for controlling seepage through pervious foundation of moderate thickness (up to about 25 m) over an impermeable bedrock formation. As can be seen from Fig. 15.15 (c), a cutoff penetrating up to 80% of the pervious depth would reduce seepage quantity only by about 50%. Therefore, to be effective, cutoffs must penetrate the full depth of the pervious foundation as shown in Fig. 15.15 (b). Such cutoffs are also advantageous in providing full-scale exploration trenches exposing all soil strata, permitting treatment of the exposed bed rock when necessary and increasing the stability of the dam because of effective replacement of the large mass of foundation soil with stronger material. The major difficulty in the construction of a rolled-earth cutoff is the dewatering of the excavated trench and keeping it dewatered until its backfilling.

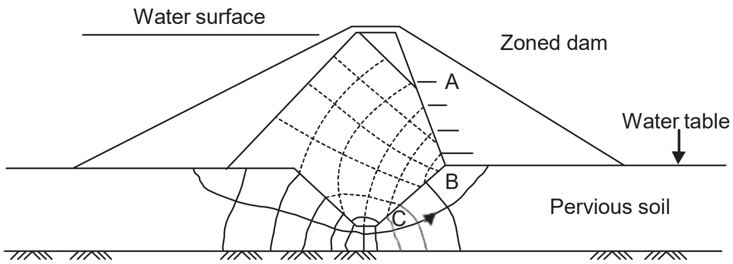
At times, the foundation is such that the average permeability of the foundation soil decreases with depth below the surface or there is a single continuous impervious layer (over other pervious layers) to which the cutoff can be connected. In such circumstances a partial vertical cutoff [Fig. 15.15 (a)] may be useful. Such partial cutoffs extending to a depth of 2 to 3 m should be specified for sites which, otherwise, may not need a seepage barrier. This provision would serve the purpose of continuous excavation through the upper layers of soil so as to better understand the subsoil conditions which, at times, may suggest the need for further excavation or provision of other suitable measures.

Slurry Trench Method

In deep alluvial deposits, provision of a rolled-earth cutoff may be expensive and difficult. In such situations, a deep narrow trench is kept open by filling it with a thick slurry of clay and bentonite. This trench, known as slurry trench, is backfilled with different types of soil, cement, and bentonite mixtures or unreinforced or reinforced concrete to act as a seepage barrier.

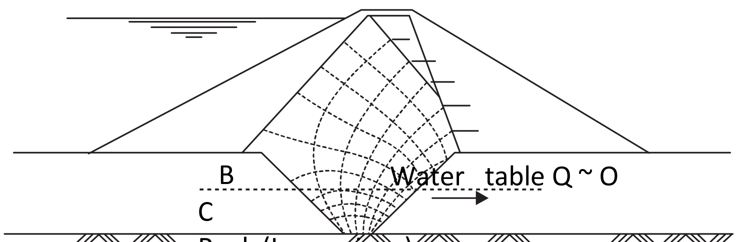
Sheet Piling

Interlocking steel sheet piling and interlocking wood are also used to construct thin cutoffs through deep alluvial foundations of embankment dams. Such thin cutoffs make the best under seepage barrier when alternative cutoff methods are costly and time-consuming. However, small openings in cutoffs and gaps at the bottom or top can result in large amount of seepage. Studies have indicated that a cutoff with 5 per cent open area at one point reduces the seepage by 60 per cent, whereas a cutoff with the same amount of open space equally divided among four openings is only 30 per cent efficient (8). Another drawback of sheet piling is the damage to itself by boulders during the driving process. Sheet piling is more effective in a homogeneous foundation than in a stratified foundation. Sheet piling is less frequently chosen because of its relatively higher cost and inherent leakage through interlocks between individual piles.



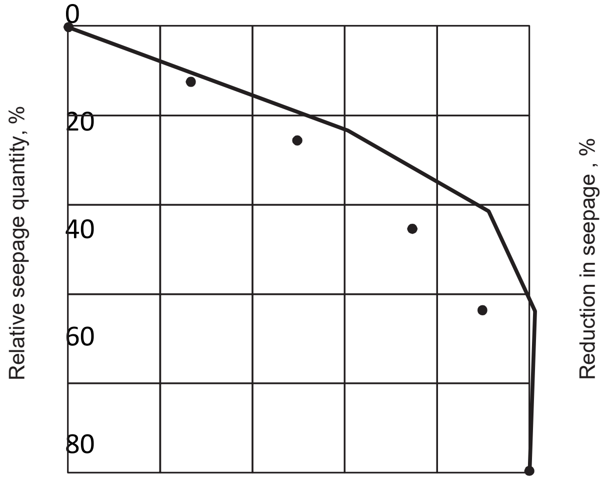
Rock (Impervious)
Cross-Section and flownet for partial cutoff

Zoned dam
A



B
Water table Q ~ O
C
Pervious soil
Rock (Impervious)
Cross-Section and flownet for complete cutoff

100 0
80 20
60 40
40 60
20 80



0 100
0 20 40 60 80 100
Percent of depth penetrated
(c) Relationship between depth of cutoff and seepage quantity

Fig. 15.15 Rolled-earth cutoff

Concrete Cutoff Walls

Concrete cutoff walls of 1.5 to 2.0 m thickness can be easily constructed by backfilling the dewatered trenches with unreinforced concrete. The trenches are braced and sheeted to keep them open. Another new method of constructing a concrete cutoff wall is by installing a continuous row of overlapping concrete piers with the help of special drilling rigs or other equipment (4).

Grouting

Grouting is the process by which fluid cement pastes are pumped through small diameter drilled holes into crevices and joints in rocks for strengthening dam foundation. After setting, the cement paste forms an impervious barrier (or grout curtain) to the seepage water. This method is also used for creating grout curtain in alluvial foundations of embankment dams. However, it is difficult to force cement grout into pores of sizes smaller than about 1.0 mm. To overcome this difficulty, chemical grouts are used in place of cement grouts. Grouting of alluvial deposits is also difficult because of: (i) the necessity of keeping the hole open with casing, (ii) the impossibility of using packers, and (iii) the lack of a proper injection technique. Chemical grouts, used until recently, were costly and not much effective either. The ''Soletanche'' method (4), however, enables effective grouting of alluvial deposits. In this method, a row of holes (125 to 250 mm diameter) at a spacing of about 3.0 m are drilled through the alluvium up to bedrock by using drilling mud and suitable drilling bits. Grout pipes (diameter about 60 mm) having circumferential rows of holes at vertical intervals of about 300 mm are set to bedrock in the drilled hole. The circumferential holes were covered with 100 mm long, tightly fitting rubber sleeves. The annular space between the grout pipe and the drilled hole is filled with a brittle clay-cement grout (Fig. 15.16). Grouting of the alluvium is then carried out from the bottom up through each of the circumferential holes in the grout pipe. The grout mix is pumped into each circumferential hole under pressure through a smaller interior grout pipe which has a set of two rubber packers spaced 300 mm apart. In this way the grout pressure can be confined to the short length of the grout pipe opposite each of the circumferential holes and the rubber sleeve.

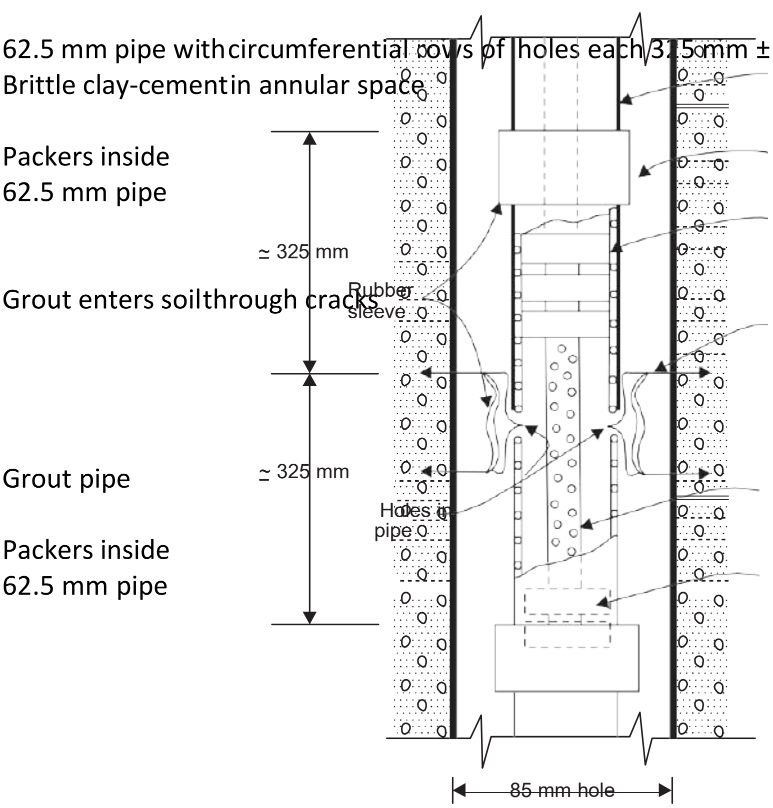


Fig. 15.16 Grouting of alluvial deposits

Grout mix is thus forced out of the rubber sleeve which expands slightly. The mix cracks the thin cylinder of clay-cement grout surrounding the pipe. The grout mix then enters the soil through these cracks. The grout mix is usually made up of clay, cement, and water in varying proportions and a predetermined amount (usually 50% of soil volume or 175% of soil voids) of the mix is pumped at each elevation.

Grout curtains may reduce seepage by significant amounts but their effect on hydrostatic pressure is relatively poor (8).

Horizontal Upstream Impervious Blanket

When the foundation soil is homogeneous and extends to large depths, a horizontal upstream impervious blanket (Fig. 15.17) offers an effective method of reducing the underseepage by increasing the length of the seepage path. Such blankets must be constructed using impervious soil in the same manner as the impervious core of the dam. At sites where a natural surface blanket of impervious soil already exists, it may be necessary only to scarify the upper surface and recompact it with proper water content and also fill the gaps to make a continuous impervious seal. The thickness and length of such blankets depend on the permeability of the blanketing material, the stratification and thickness of the pervious foundation, and the reservoir depth. Thickness ranging from 0.6 to 3 m is generally used. If the blanket material is very impervious compared to the natural foundation soil so that the seepage through the blanket is negligible, the length of the blanket can be related to the reduction in underseepage. Considering Fig. 15.17, the underseepage Q in the absence of the blanket is given as

$$Q = K_f \frac{H}{x_d} Z_f \qquad (15.17)$$

in which K_f is the coefficient of permeability of the foundation soil, and Z_f is the thickness of the foundation. If the provision of the blanket of length x reduces the underseepage to pQ, then

$$pQ = K_f \frac{H}{x_d + x} Z_f \qquad (15.18)$$

Combining Eqs. (15.17) and (15.18)

$$p = \frac{x_d}{x_d + x} \qquad (15.19)$$

or

$$x = x_d \frac{(1 - p)}{p} \qquad (15.20)$$

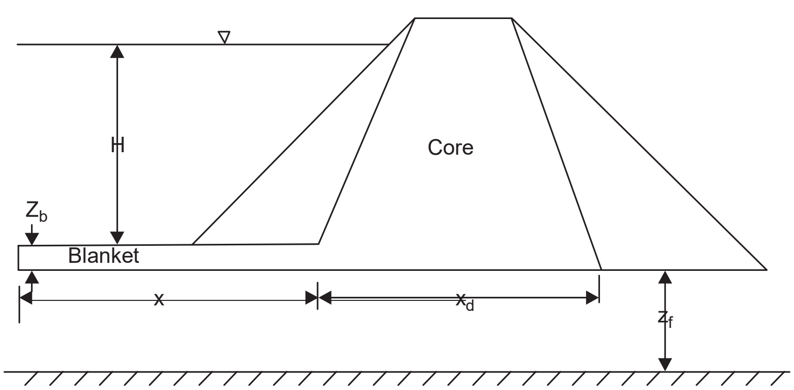


Fig. 15.17 Horizontal upstream impervious blanket

It may, however, not always be possible to have an impervious blanket through which the seepage can be considered negligible. In such cases, there is an optimum length of the blanket and there is no significant reduction in the underseepage with further increase in the length of the blanket. The effectiveness of such blankets (Fig. 15.18) is analysed by using the following Bennett's fundamental differential equation (10):

$$\frac{d^2 h}{dx^2} = a^2 h \qquad (15.21)$$

where, h is the head loss through the blanket which equals the difference of the head on the two sides of the blanket under which percolation takes place and a is given as:

$$a^2 = \frac{K_b}{K_f\, Z_f\, Z_b} \qquad (15.22)$$

Here, K_f and K_b are coefficients of permeability of the foundation, and the blanket material, respectively. For a blanket of constant thickness Z_b and infinite horizontal extent, Bennett (10) obtained

$$x_r = \frac{1}{a} \qquad (15.23)$$

where, x_r is termed the effective length of the blanket and is defined as the length of the prism of foundation which will be able to carry the actual foundation seepage under the same head loss h_0 (up to the end of the blanket) with a linear hydraulic gradient, i.e.,

$$q = K_f \frac{h_0}{x_r} Z_f \qquad (15.24)$$

For the case of blanket of constant thickness and finite length (i.e., the part of the foundation seepage enters from the reservoir bed upstream of the blanket), Bennett (10) obtained

$$x_r = \frac{e^{2ax} - 1}{a\,(e^{2ax} + 1)} = \frac{1}{a} \tan h\,(ax) \qquad (15.25)$$

Here, x_r represents the effective length of the blanket up to a distance x downstream from its upstream end (x = 0). If L is the total length of the blanket, then its effective length is given as

$$x_r = \frac{1}{a} \tan h\,(aL) = \frac{e^{2aL} - 1}{a\,(e^{2aL} + 1)} \qquad (15.26)$$

F.R.L.

h h_o

Foundation HGL

Z_b H

d_x

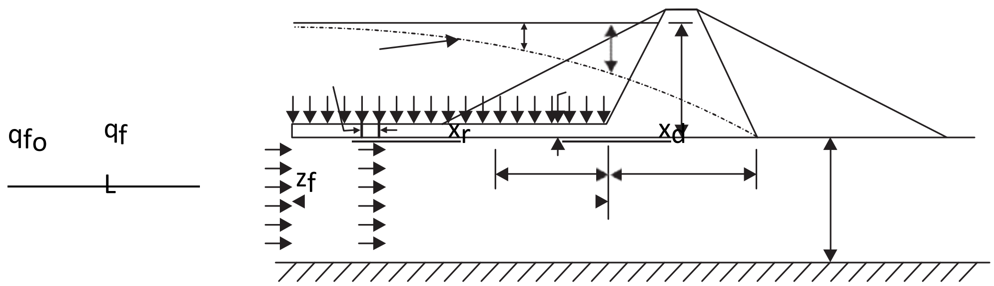


Fig. 15.18 Definition sketch for Bennett's analysis

Having determined the effective length of the blanket using a suitable equation [Eq. (15.23) for infinite length and Eq. (15.26) for finite length], one can determine the reduction in the foundation seepage as follows:

If x_d is the base width of the impervious core of the dam, the foundation seepage is given as

$$q_{f_1} = K_f \frac{H}{x_d} Z_f \quad (15.27)$$

When an impervious blanket, whose effective length is x_r, is available, the foundation seepage is given as

$$q_{f_2} = K_f \frac{H}{x_r + x_d} Z_f \quad (15.28)$$

Here, H is the total head. Also, the loss of head up to the end of the blanket will be equal to

$$H \frac{x_r}{x_r + x_d}$$

The optimum length of an upstream impervious blanket x_o is given (10) by the expression $a x_o = \sqrt{2}$. Length of the blanket beyond x_o will increase its effective length x_r only marginally.

Zoning

Most often embankment dams and particularly earth dams have an internal impervious core surrounded on both sides by outer sections called shell. The core provides water tightness to the embankment. The core can either be a thin sloping core, [Fig. 15.2 (a)], or a somewhat thicker sloping core, [Fig. 15.2 (b)] or even a thick central core [Fig. 15.2 (c)]. Impervious core dissipates hydrostatic energy rapidly but increases the chances of piping due to the presence of large hydraulic gradients. Therefore, at the boundaries of the core and shells, filters must always be used. Whenever sufficient quantity of highly impermeable soils are readily available, a thick central core type embankment [Fig. 15.2 (c)] should be constructed. A thick core provides wide zone for energy dissipation, relatively low hydraulic gradients, and longer contacts with foundations. Dams having extremely narrow cores induce large hydraulic gradients within the embankment.

Downstream Drains

Dams with heights of more than about 6 m should always be provided with some type of downstream drain which is constructed using material many times more pervious than the embankment soil. Such drains reduce the pore water pressures in the downstream shell and, thus, increase the stability of the downstream slope against sliding. These downstream drains could be in the form of toe drains, [Fig. 15.1 (a)], horizontal blanket drains [Fig. 15.1 (b)], chimney drains [Fig. 15.1 (c)], strip drains [Fig. 15.19] or partially penetrating toe drains [Fig. 15.20] depending upon the height of the dam, the cost and availability of previous material, and the permeability of the foundation.

Horizontal Blanket Drains: These drains [Fig. 15.1 (b)] are widely used for dams of low to moderate heights and are highly effective only on relatively uniform foundations and in non-stratified embankments. Such drains collect seepage from the embankment as well as the downstream portion of the foundation and accelerate the consolidation of the foundation. Such drains are, however, not suitable for stratified embankments. These drains usually extend from the downstream toe to the upstream ranging in length from 25 to 100 per cent of the projected length of the downstream slope. The thickness of such drains should always be more than 1 m and should be sufficient to be able to carry the maximum anticipated seepage with a conservative factor of safety. The drain would consist of relatively coarse particles and should, therefore, be surrounded by suitable filter layers to prevent the migration of finer material of embankment or foundation. At sites where pervious material is available in small amount, the general effect of horizontal blanket can still be obtained by constructing strip drain (Fig. 15.19).

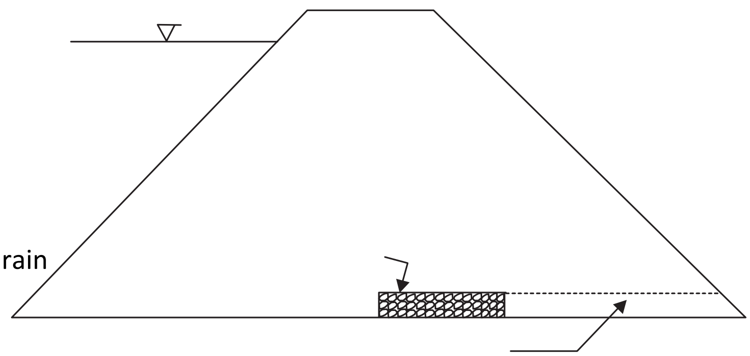


Fig. 15.19 Strip drain

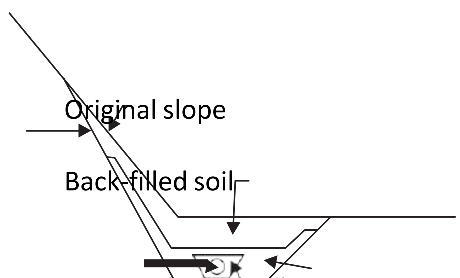


Fig. 15.20 Partially penetrating toe drain

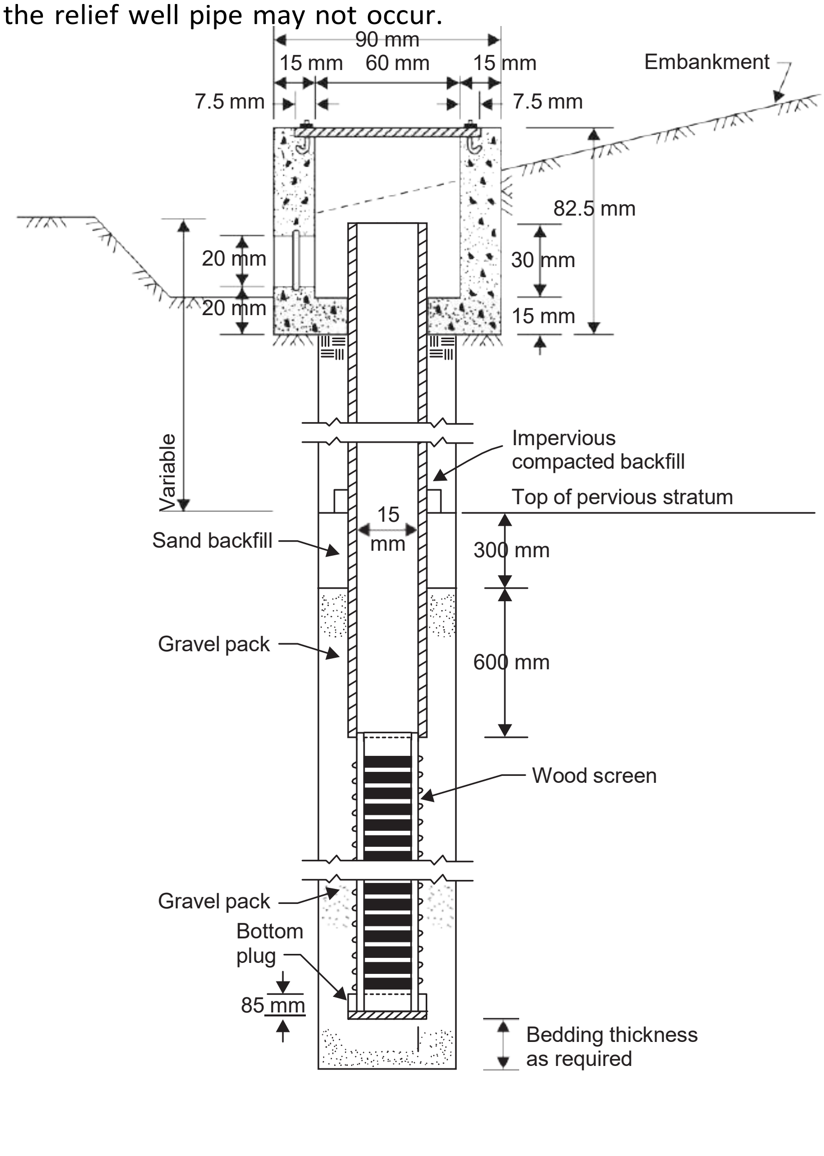
Toe Drains: If sufficient quantity of boulder or quarried rock is available, a toe drain (orrock-toe) [Fig. 15.1 (a)] of height about 0.25 to 0.35 times the height of the embankment will be very effective in controlling seepage from stratified embankments. The rock-toe also protects the lower part of the downstream slope from tail-water erosion. Generally, the inner slope ofrock-toe is provided at 1 : 1.

Chimney Drains: Chimney drains [Fig. 15.1 (c)] combine the advantages of horizontal blanket drains as well as toe drains. These drains can completely intercept embankment seepageirrespective of the extent of embankment stratification. Besides, these drains keep the seepage line well below the downstream face. For effective functioning, these drains and their outlets must have sufficient permeability to discharge seepage water without building up excessive head.

Partially Penetrating Toe Drains: If undesirable foundation seepage conditions are likely to develop during the operation of a dam, a partially penetrating toe drain (Fig. 15.20) should be constructed so as to improve the seepage conditions. The perforated pipes are connected to gravity outlets. These drains will not be effective when the foundation is such that the drains are separated from underlying pervious strata by impervious layers. In such cases, relief wellsare more suitable.

Relief Wells

Relief wells (Fig. 15.21) are provided when pervious strata of embankment foundation are too deep to be penetrated by rolled-earth cutoffs or toe drains. Relief wells can penetrate most pervious water-bearing strata and relieve uplift pressures effectively. Spacing between the relief wells should be small enough to lower the water pressure to the desired safe level. Relief wells must penetrate through the complete depth of the pervious foundation, if possible. A relief well should preferably have an interior perforated pipe (i.e., the well screen) with a minimum diameter of 15 cm or larger, if heavy foundation seepage is anticipated. The annular space surrounding the well screen is backfilled with graded filter. However, near the surface, the annular space is backfilled with impervious soil or concrete so that upward flow of water outside the relief well pipe may not occur.



15
mm

Fig. 15.21 Gravael-packed relief well screen (4)

Example 15.3 Using Bennett's solution for upstream impervious blanket, find the discharge through the foundation of an earth dam having a central core (Fig. 15.22) for the following data:

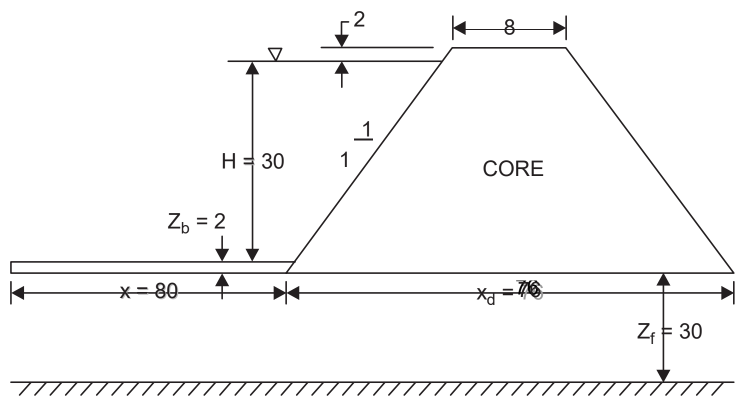
Water depth over upstream blanket = 30.0 m Crest width = 8.0 m

Freeboard = 2.0 m

Upstream and downstream slopes of the core = 1 : 1 Thickness of the impervious blanket = 2.0 m Length of the blanket (connected to the core) = 80.0 m Depth of pervious foundation = 30.0 m Coefficient of permeability of

foundation soil= 2×10^{-4} m/s

core and blanket soil = 2×10^{-6} m/s



All dimensions in metres

Fig. 15.22 Sketch for Example 15.3

Solution: Using Eq. (15.22)

$$a = \sqrt{\frac{K_b}{K_f Z_f Z_b}} = \sqrt{\frac{2 \times 10^{-6}}{2 \times 10^{-4} \times 30 \times 2}} = 0.0129$$

Effective length of blanket,

$$x_r = \frac{1}{a} \tanh(ax)$$

$$= \frac{1}{0.0129} \tanh(0.0129 \times 80) = 60.0 \text{ m}$$

From Eq. (15.28),

$$q_f = K_f Z_f \frac{H}{x_r + x_d} = 2 \times 10^{-4} \times 30 \times \frac{30}{60 + 76}$$

$= 1.324 \times 10^{-3}$ m^3/s/m

Filters

In a homogeneous embankment, the individual soil particles subjected to seepage forces cannot move because they are held in place by the neighbouring soil particles. But, at the boundaries between soils of different sizes, the finer soil particles may be washed into the void spaces of the coarser material. To prevent such migration of soil particles, it should always be ensured that the relative gradation of adjacent soil zones meet established filter criteria. If the difference in size of soils of the fine and coarse zones of an embankment is too large to satisfy the filter criteria, zones of intermediate gradation, known as filters must always be provided in between the fine zone and coarse zone.

There are two main conflicting requirements of a filter layer:

the filter layer must be more pervious than the protected soil so that the filter acts as a drain, and

the size of the particles used in the filter layer must be small enough to prevent migration of the particles of the protected soil into the voids of the filter layer.

Further, the filter layer should be sufficiently thick to provide good distribution of all particle sizes throughout the filter (11). The following rules are widely used for the design offilter layer (4, 11):

$$\frac{D_{15} \text{ of the filter}}{D_{15} \text{ of the protected soil}} \;\square\; 5$$

$$\frac{D_{15} \text{ of the filter}}{D_{85} \text{ of the protected soil}} \;\square\; 5$$

$$\frac{D_{50} \text{ of the filter}}{D_{50} \text{ of the protected soil}} \;\square\; 25$$

The gradation curve of the filter should have approximately the same shape as the gradation curve of the protected soil.

Where the protected soil contains a large percentage of gravels, the filter should be designed on the basis of the gradation curve of the portion of the material which is finer than the particles passing one-inch sieve.

Filters should not contain more than about 5% of fines passing no. 200 sieve, and the fines should be cohesionless.

Here, D_{15}, D_{50}, and D_{85} represent the particle sizes which are, respectively, coarser than the finest 15, 50, and 85 per cent of the soil, by weight. These filter criteria are based on studies with non-cohesive soils and take into consideration only the grain size of the protected soil. As such, these rules may be conservative, particularly for clays which can resist piping action because of cohesion.

Theoretically, the required thickness of a horizontal filter is very small—about 15 cm for sand and 30 cm for gravel. But, from practical considerations, a minimum thickness of 1.0 m is desirable (11). For vertical or inclined filters, the minimum width of filters is about 2 to 3 m for convenience in construction.

STABILITY ANALYSIS OF EMBANKMENT DAMS

Quantitative assessment of the stability of slopes is very important in the design of embankment dams. Prior to 1935, the side slopes of embankment dams were selected purely on the basis of past experiences and preferences of designers. During the last fifty years there has been considerable improvement in the understanding of soil shear strength, laboratory methods of soil testing, and the methods of stability analysis. As a result of all these developments, it is now possible to quantitatively assess the stability of slopes of an embankment dam.

Critical Conditions for the Stability of an Embankment Dam

The following conditions are considered critical for the stability of an embankment dam (12).

During Construction With or Without Partial Pool

When an embankment is constructed, water is added to soil during compaction. This increases pore pressures and may cause failure of both upstream (if there is no water in the reservoir) and downstream slopes. During construction, the reservoir is usually empty and, hence, both the upstream and downstream slopes need to be analysed. The magnitude and the distribution of the construction pore pressures depend primarily on the construction moisture content in the embankment, natural moisture content in the foundation, soil properties, construction rate, dam height, and internal drainage. Since high construction pore pressures exist only during the first few years of the life of the dam (when reservoir may not have been filled completely), more conservative and expensive design is avoided and, instead, suitable steps are taken to reduce the construction pore pressures. These steps include:

compacting impervious core at an average moisture content which is 1–3 per cent below the optimum moisture content,

making impervious core section thinner,

providing internal drains within impervious core section to accelerate pore pressure dissipation,

accepting a lower factor of safety on the plea that a slide during construction would not cause catastrophic failure and only delay the completion of the dam, and

having longer construction period so that some pore pressure is dissipated from the already-constructed fill before laying another layer.

For analysing the stability of slopes during construction, one needs information aboutpore pressure. As per USBR's simple approach, the pore pressure head during construction equals, approximately, 1.25 times the height of the fill. Alternatively, one can measure pore pressures in sealed laboratory specimens which have been compacted and subjected to increasingstresses simulating the field conditions in respect of the water content, densities and increasing stresses, and use them for analysis. However, there can be some difficulties in simulation. If pore pressures are allowed to dissipate during construction by causing break in construction,the pore pressures are estimated (12) by using the following Bishop's method which uses Hilf's equation.

Bishop's method requires the knowledge of volume change – effective stress relationship [($\Delta V/V$) v/s σ'] for the compacted fill. This is determined experimentally by testing representative specimens. The volume change – pore pressure relationship [($\Delta V/V$) v/s Δu] is obtained for each stage of construction using Boyle's law and Henry's law (for solubility of air in water).

Any sample of compacted fill having a volume of V_o , Fig. 15.23 includes volume of soil grains V_s and volume of voids V_v which equals the sum of the volume of water V_w and volume of air V_a occupying the voids. The volume of air dissolved per unit volume of water is defined as Henry's coefficient of solubility, H whose approximate value is 0.02. The degree of saturation S_o is the ratio of volume of water and volume of voids. Thus,

Initial volume of free air $= V_a$ + dissolved air(at initial pressure p_o) $= (V_v - V_w) + HV_w$

$= V_v - V_v S_o + HS_oV_v$

$= V_v(1 - S_o + S_oH)$

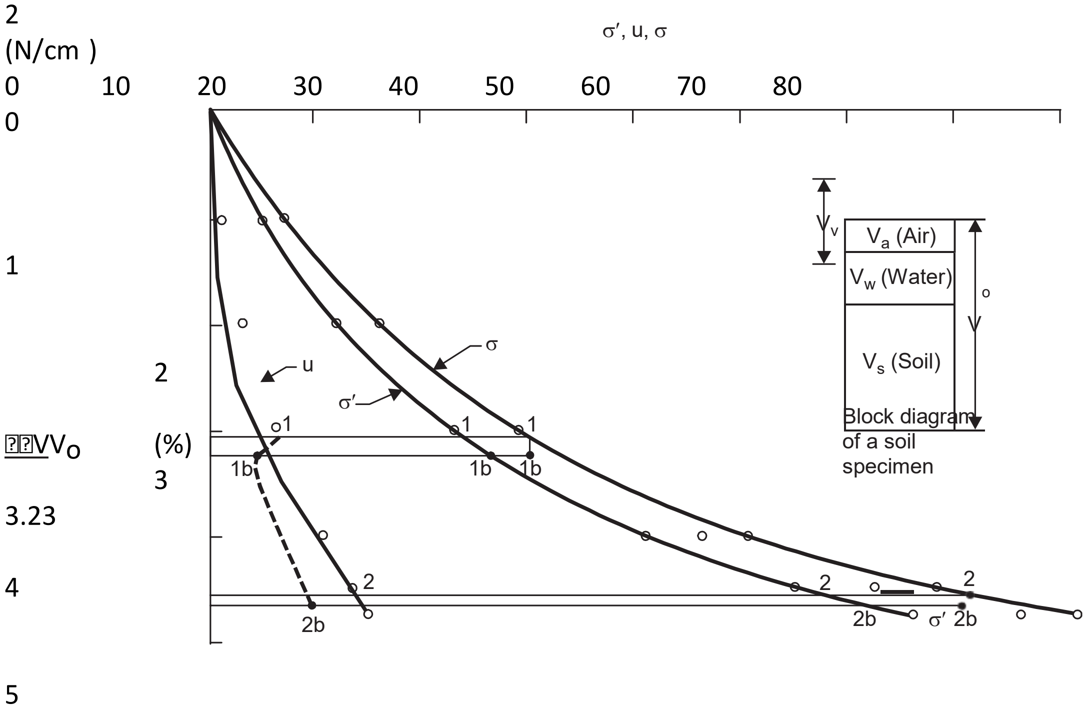


Fig. 15.23 Variation of u, σ′ and σ with $\Delta V/V_o$

At a new pressure p, the volume of air, using Boyle's law, is, therefore, given as

$$\frac{p_o}{p} V_v (1 - S_o + S_o H)$$

Therefore, change in volume of air, $\Delta V = (p_o/p)\, V_v (1 - S_o + S_o H) - V_v (1 - S_o + S_o H)$

$$p\,\{\Delta V + V_v (1 - S_o + S_o H)\} = p_o V_v (1 - S_o + S_o H)$$

$$\frac{p_o}{p} = 1 + \frac{\Delta V/V_v}{1 - S_o + S_o H} = 1 + \frac{\Delta V/n_o V_o}{1 - S_o + S_o H}$$

in which n_o is the initial porosity of soil.

$$1 - \frac{p_o}{p} = -\frac{\Delta V/V_o}{n_o (1 - S_o + S_o H)}$$

$$p - p_o = -\frac{(\Delta V/V_o)\, p}{n_o (1 - S_o + S_o H)} = -\frac{(\Delta V/V_o)\, p_o}{n_o (1 - S_o + S_o H)\,(p_o/p)}$$

$$= -\frac{(\Delta V/V_o)\, p_o}{n_o (1 - S_o + S_o H)\left[1 + \dfrac{\Delta V/V_o}{n_o (1 - S_o + S_o H)}\right]}$$

$$= -(\Delta V/V_o)\, p_o$$

$\underline{?V}\ ?\ n_o\ (1\ ?\ S_o\ ?\ S_o\ H)V_o$

If the initial pressure p_o is the atmospheric pressure, p_a then change in pore pressure, $\Delta u = p - p_o = p - p_a$.

Therefore,
$$\Delta u = -\frac{\left(\dfrac{\Delta V}{V_o}\right) p_a}{\dfrac{\Delta V}{V_o} + n_o(1 - S_o + S_o H)} \qquad (15.29)$$

Equation (15.29) is known as Hilf's equation. This equation is applicable under the following conditions:

There is only vertical compression and no lateral bulging of the fill.

Pressures in pore water and pore air are the same. Strictly speaking, Hilf's equation gives pore air pressure (say, U_a) whereas the effective stress depends on pore water pressure (say, U_w) which equals the sum of U_a and capillary pressure U_c which is negative for unsaturated condition and zero for saturated condition. Since earth fills are usually placed at 80 to 90 per cent degree of saturation, the capillary pressure is usually small and, therefore, neglected.

Decrease in volume of fill is due to compression of pore air and dissolution of pore air in pore water.

There is no dissipation of pore pressure during construction.

Boyle's and Henry's laws are applicable.

Further, when all the air in pores has gone into solution,

$$\Delta V = V_a = V_v - V_w = V_v - V_v S_o = V_v (1 - S_o)$$

$$\therefore \frac{\Delta V}{V_o} = \frac{V_v (1 - S_o)}{V_o}$$

$$= n_o(1 - S_o) \text{ with negative sign, of course.}$$

Thus,

$$\Delta u = \frac{n_o (1 - S_o)\, p_a}{n_o (1 - S_o + S_o H) - (1 - S_o) n_o}$$

$$= \frac{(1 - S_o)\, p_a}{S_o H} = \frac{(1 - S_o) V_v\, p_a}{H V_w}$$

and,
$$\Delta u = \frac{p_a V_a}{H V_w} \qquad (15.30)$$

when all the air in pores has gone into solution, the soil is fully saturated and any further increase in load ($\Delta\sigma$) results in equal increase in pore pressure (Δu) as there is no air to get compressed.

In order to prepare total stress versus pore pressure relationship for an earthfill being constructed in stages, following steps are followed:

Using experimental data, plot $\Delta\sigma$ against $\Delta V/V_o$, Fig. 15.23.

For different assumed values of $\Delta V/V_o$, compute Δu using Hilf's equation, Eq. (15.29) and the total stress $\sigma = \sigma' + \Delta u$. Carry out these computations till either $\sigma = \rho g H_1$ or

$[\Delta V/V_o] = n_o\,(1 - S_o)$ i.e., when all pore air has gone into solution. Here, H_1 is the height of the dam (or earthfill) at the end of the relevant stage of construction.

Plot σ v/s $\Delta V/V_o$ and Δu v/s $\Delta V/V_o$, Fig. 15.23.

Let the pore pressure Δu at the end of the first stage be dissipated by x per cent before the beginning of the second stage.

Pore pressure at the beginning of the second stage

$$\Delta u_{1b} = \left(1 - \frac{x}{100}\right)\Delta u_1$$

$$\sigma'_{1b} = \sigma'_1 + \frac{x}{100}\,\Delta u_1$$

and $$\sigma_{1b} = \sigma'_{1b} + \left(1 - \frac{x}{100}\right)\Delta u_1 = \rho g H_1$$

Also, $p_{ob} = p_o + \Delta u_{1b}$ (in absolute terms)

With absolute pore pressure p_{ob}, Hilf's equation can again be used to compute rise in pore pressure with further loading for the second stage of loading provided that the values of S_o and n_o are recalculated at 1b. During the loading stage under undrained conditions, the degree of saturation S can be expressed as follows:

$$S = \frac{\text{Initial volume of water}}{\text{volume of voids}} = \frac{S_o V_v}{V_v - \Delta V} = \frac{S_o}{1 - \dfrac{\Delta V}{V_v}}$$

Since $$\Delta V = \frac{p_o}{p} V_v (1 - S_o + S_o H) - V_v (1 - S_o + S_o H)$$

$$\frac{\Delta V}{V_v} = (1 - S_o + S_o H)\left(\frac{p_o}{p} - 1\right)$$

For the beginning of the second stage, the degree of saturation S_{ob} corresponding to new pore pressure (gauge) p_{ob} is, therefore, expressed as:

$$S_{ob} = \frac{S_o}{1 - \left(\dfrac{p_o}{p_{ob}} - 1\right)(1 - S_o + S_o H)}$$

Similarly, new value of porosity at stage 1b,

$$n_{ob} = \frac{\text{New volume of voids}}{\text{Initial volume}} = \frac{V_v - \Delta V_{1b}}{V_o} = n_o - \frac{\Delta V_{1b}}{V_o}$$

= n_o G J

F V l 1b o

Here, $\left(\frac{\Delta V}{V_o}\right)_{1b}$ is the value of $\Delta V/V_o$ at 1b and is negative.

Here, it has been implicity assumed that there is no drainage during the construction (loading stage) and drainage is only during the periods when construction is stopped temporarily for dissipation of pore pressure. However, there will be some drainage during the construction which can be accounted for by applying a suitable dissipation factor at the end of each step of construction.

Using now 1b as origin [i.e., $\Delta u = 0$ and $(\Delta V/V_o) = 0$], plot Δu v/s $\Delta V/V_o$ and σ v/s $\Delta V/V_o$ using steps (ii) and (iii) with new values of porosity n_{ob} and degree of saturation, S_{ob} computed in step iv.

Likewise, computations can be carried out for further stages of construction, if any.

Plot σ versus u on a separate graph sheet from the above results,

Above steps of computations have been illustrated in the following example.

Example 15.4 A fill is placed at an initial saturation of 80% (S_o) and initial porosity of 37.5% (n_o). The average unit weight of the compacted fill is 20,000 N/m^3. The dam is raised to a height of 15 m in the first stage after which there is a gap during which one-third of pore pressure will have dissipated. In the next stage, the dam is raised to a height of 35 m. Plot u v/s σ. The variation of $\Delta V/V_o$ v/s $\Delta\sigma'$ is as follows:

$\Delta V/V_o$(%)	1	2	3	4	4.5	4.75
$\Delta\sigma'$ (N/cm^2)	5.0	12.5	23	40	54.5	66.0

Solution: During the first stage:

$$\Delta u = -\frac{\left(\frac{\Delta V}{V_o}\right)p_o}{\left(\frac{\Delta V}{V_o}\right) - n_o(1 - S_o + S_o H)}$$

with p_o = atmosheric pressure = 10.3 N/cm^2 and H = 0.02

$$= -\frac{10.3\left(\frac{\Delta V}{V_o}\right)}{\frac{\Delta V}{V_o} - 0.375\,(1 - 0.80 + 0.80 \times 0.02)} = \frac{10.3\left(\frac{\Delta V}{V_o}\right)}{0.0798 - \left(\frac{\Delta V}{V_o}\right)}$$

Using this equation and the curve $\frac{\Delta V}{V_o}$ v/s $\Delta\sigma'$, the following table can be prepared.

$\Delta V/V_o$(%)	Δu	$\Delta\sigma'$	σ
1	1.48	5.0	6.48
2	3.45	12.5	15.95
3	6.21	23.0	29.21
4	10.35	40.0	50.35

4.5	13.32	54.5	67.82
4.75	15.15	66.0	81.15

At the end of the first stage of construction (1 on Fig. 15.23)

$\sigma = \gamma H = 20000 \times 15 \text{ N/m}^2 = 300000 \text{ N/m}^2$

$\sigma = 30 \text{ N/cm}^2$

For $\sigma = 30 \text{ N/cm}^2$

$\frac{\Delta V}{V_o} = 3.06\%$ from graph

$\therefore \quad \sigma' = 23.3 \text{ N/cm}^2$ from graph

and $\quad u = 6.7 \text{ N/cm}^2$ from graph

After one-third of pore pressure dissipation (= 2.23 N/cm^2)

$u = 4.47 \text{ N/cm}^2$; $\sigma' = 23.30 + 2.23 = 25.53 \text{ N/cm}^2$;

$\sigma = 30 \text{ N/cm}^2$ and $\Delta V/V_o = 3.23\%$ (from curve)

At the beginning of the 2nd stage of construction, (1b on Fig. 15.23)

$(\Delta V_{1b}/V_o) = 3.23\%$; $u_{1b} = 4.47 \text{ N/cm}^2$, $\sigma'_{1b} = 25.53 \text{ N/cm}^2$ and $\sigma_{1b} = 30 \text{ N/cm}^2$

$p_{ob} = p_o\ (= p_{atm}) + u = 10.3 + 4.47 = 14.77 \text{ N/cm}^2$

$$S_{ob} = \frac{S_o}{1 + \left(\frac{p_o}{p_{ob}} - 1\right)(1 - S_o + S_o H)} = \frac{0.80}{1 + \left(\frac{10.3}{14.77} - 1\right)(1 - 0.8 + 0.8 \times 0.02)} = 0.853$$

$$n_{ob} = n_o + \frac{\Delta V_{1b}}{V_o} = 0.375 - 0.0323 \left(\text{as } \frac{\Delta V_{1b}}{V_o} \text{ is − ve}\right)$$

$= 0.343$

Using these values and Hilf's equation, one obtains

$$\Delta u_{12} = -\frac{\left(\frac{\Delta V_{12}}{V_o}\right) p_{ob}}{\left(\frac{\Delta V_{12}}{V_o}\right) + n_{ob}(1 - S_{ob} + S_{ob} H)}$$

$$= -\frac{14.77\left(\frac{\Delta V_{12}}{V_o}\right)}{\frac{\Delta V_{12}}{V_o} + 0.343(1 - 0.853 + 0.853 \times 0.02)}$$

$14.77\left(\frac{\Delta V_{12}}{V_o}\right)$

$= - \square V_{12} \square 0.056 .$

V_o

One can now compute Δu_{12} and prepare the following table :

$\frac{\Delta V}{V_o}$ (%)	$\frac{\Delta V_{12}}{V_o} = \frac{\Delta V}{V_o} - \frac{\Delta V_{1b}}{V_o} = \frac{\Delta V}{V_o} - 0.0323$	Δu_{12} (w.r.t. 1b as origin) (N/cm^2)	Pore pressure (w.r.t. initial pore pr.) $= u_{1b} + \Delta u_{12}$ (N/cm^2)	σ' (N/cm^2)	σ (N/cm^2)
3.23	0	0	4.47	25.53	30.00
4.0	0.0077	2.35	6.82	40.00	46.82
4.5	0.0127	4.33	8.80	54.50	63.30
4.65	0.0142	5.02	9.49	60.61	70.00

The values of Δu_{12} are shown plotted in Fig. 15.23 from 1b to 2b. Using this figure, a plot of u versus σ can be prepared, Fig. 15.24.

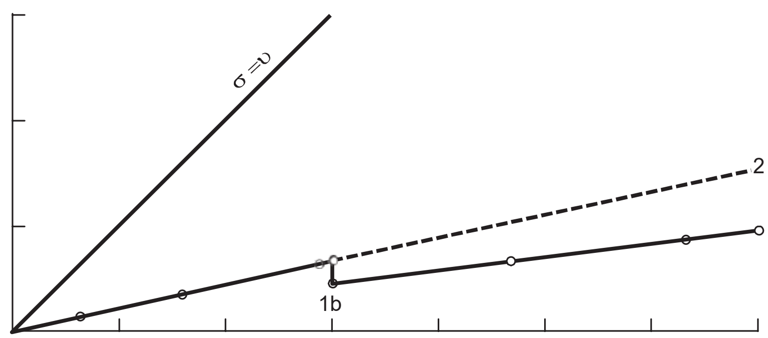


Fig. 15.24 Variation of u with σ

Partly-filled Reservoir Condition

Soon after the construction of an embankment dam, it takes a few seasons to fill the reservoir. This means that the dam will experience different amounts of pore pressures at different times depending upon the reservoir level. Since the upstream slope would not be fully covered with water during partly-filled reservoir condition, it needs to be examined for stability for various reservoir levels ranging from one-third to two-thirds the height of the full reservoir head, and the minimum factor of safety is worked out.

Sudden Drawdown Condition

This condition corresponds to lowering of reservoir level at a rate much faster than the rate of subsequent dissipation of pore pressure. The condition results in excess pore pressures and unbalanced seepage forces and may become worse when the materials of the upstream portion of the dam are not freely draining. If the coefficient of permeability of the shell material is less than 10^{-4} cm/s, full pore pressure should be considered (12). If the coefficient of permeability of the shell material is greater than 10^{-2} cm/s, pore pressure may not be considered in the analysis. For other values of the coefficient of permeability of the shell material, pore pressure values may be interpolated (12). These recommendations are based on a drawdown rate of 3 m/month.

The drawdown pore pressure in clay core (Fig. 15.25) can be determined using Bishop's formula (12):

$U = \rho g\,[h_c + h_r\,(1 - n) - h]$ (15.32)

Here, U is the drawdown pore pressure at any point within the core, ρ the mass density of water, g the acceleration due to gravity, h_c the height of core material at the point under consideration, h_r the height of the shell material at that point, h the drop in the head under steady seepage condition at the point, and n is the specific porosity of the shell material, i.e., the volume of water draining out from the shell per unit volume.

The resisting and driving forces for impermeable material are calculated using the submerged and saturated weight, respectively, if the pore pressures are not otherwise included in the stability analysis (13).

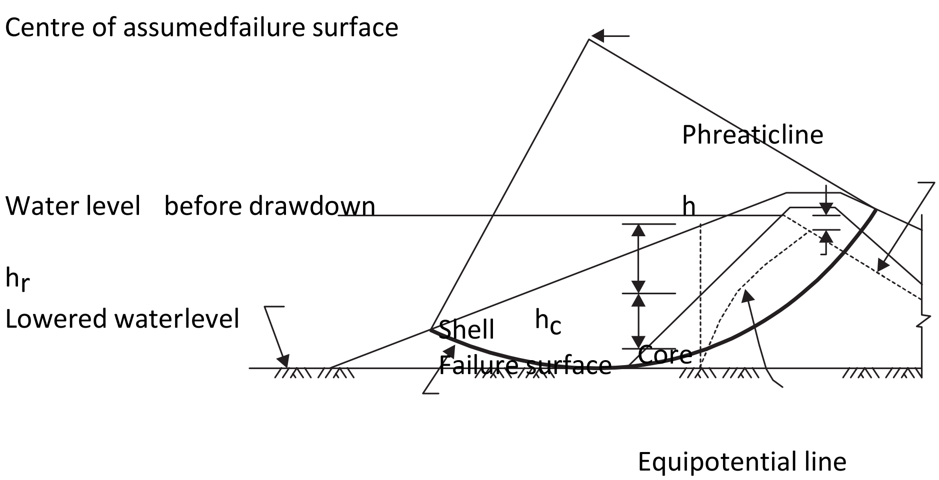


Fig. 15.25 Downstream pore pressure in clay core

Steady Seepage at Full Reservoir Condition

This condition develops when the reservoir is full and as such any shear slide would lead to a disastrous failure. Therefore, the stability analysis for this condition must be carried out more conservatively. The stability of the downstream slope should be analysed for this condition using the effective stress method and taking into account pore pressures owing to gravity flow. Pore pressures on account of changes in the embankment volume are not considered in the analysis, because the shear strains imposed on a well-constructed embankment are likely to dilate (i.e., expand the volume) the soil and reduce the pore pressures temporarily (13).

The following unit weights may be used for the calculation of driving and resisting forces when pore pressures are otherwise not included in the stability analysis (13).

Location	Driving force	Resisting force
Below phreatic surface	Saturated weight	Submerged weight
Above phreatic surface	Moist weight	Moist weight

Steady Seepage with Sustained Rainfall

If the downstream shell is relatively pervious, the pore pressures can increase appreciably by the penetration of rain water. In such situations, the gravity flownet should be constructed assuming that the dam crest and the downstream slope are sources of supply for seepage water (4) (Fig. 15.26). The downstream slope is analysed for stability assuming that partial saturation occurs due to rainfall. The saturation for the downstream shell is taken as 50 per cent (for coefficient of permeability of the shell less than 10^{-4} cm/s and 0 per cent for coefficient of permeability of the shell more than 10^{-2} cm/s) or a suitable value between 0 and 50 per cent depending upon the value of the coefficient of permeability of the shell material (12).

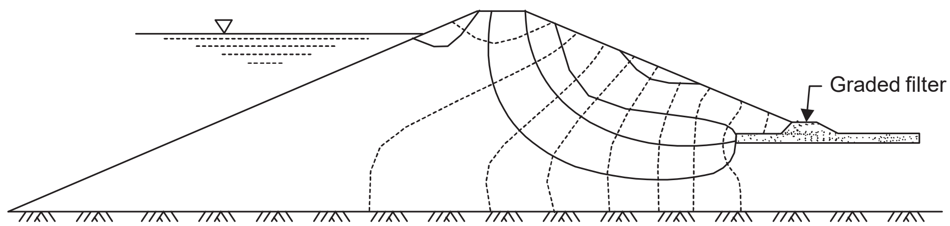


Fig. 15.26 Flownet for the downstream shell of an embankment dam during sustained rainfall

Earthquake Condition

The shaking of dam during an earthquake may cause most adverse conditions of stability of an embankment dam. For example, due to the shaking of the embankment, cracks within the core may widen causing increased leakage and consequent piping failure. This shaking may also result in the settlement of the crest due to the compression of the foundation and embankment. Acceleration forces acting on the embankment dam during an earthquake may cause shear slide of the slopes of the dam. Both the upstream and downstream slopes must be examined for earthquake effects if the dam is situated in an earthquake-prone region. Besides, other suitable measures to prevent earthquake damage are also adopted. Such measures include provision of graded filter downstream of the core, extra freeboard, very pervious downstream zones for disposing of the maximum anticipated leakage rapidly, flatter slopes near the top of the dam, better foundation treatment, and so on.

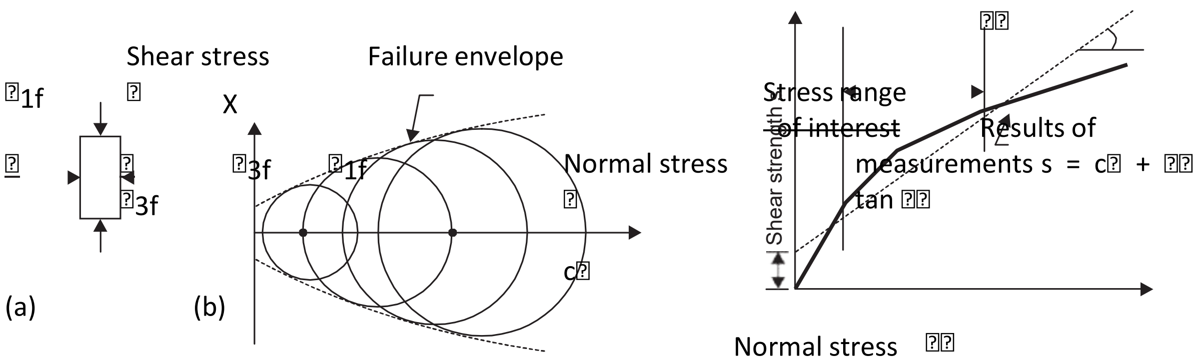
Shear Strength of Soils

Soils derive their strength from contact between particles capable of transmitting normal aswell as shear forces. The contact between soil particles is mainly due to friction and the corresponding stress between the soil grains is called the effective (or intergranular) stress σ'. Thus, the shear strength of a soil is mainly governed by the effective stress. Besides the effective stress between soil grains, the pore water contained in the void spaces of the soil also exerts pressure which is known as pore pressure, u. The sum of the effective stress and pore pressure acting an any given surface within a compacted earth embankment is called the total stress σ. As the pore water cannot resist shear, all shear stresses are resisted by the soil grains only. The effective stress is approximately equal to the average intergranular force per unit area and cannot be measured directly (14). The total stress is equal to the total force per unit areaacting normal to the plane. The pore water affects the physical interaction of soil particles. Soils with inactive surfaces do not absorb water. But, clay particles, formed of silicates which are, frequently, charged electrically, absorb water readily and exhibit plastic, shrinkage, and swelling characteristics. Besides, a change in pore pressure can directly affect the effective stress. The pore water thus influences the shear strength parameters of a soil to a considerable extent.

The shear strength of a soil is fully mobilised when a soil element can only just support the stresses exerted on it. For most soils, the shear strength of any surface, at failure, is approximated by the following Mohr-Coulomb's linear relationship [Fig. 15.27]:

$$s = c' + \sigma' \tan \phi' \qquad (15.31)$$

where, s = shear strength of soil (or shear stress at failure) on the surface under consideration,
c' = cohesion,
ϕ' = angle of internal friction (or angle of shearing resistance), and
σ' = effective normal stress acting on the failure surface = σ – u. Here, c' and ϕ' are determined using effective stresses.



(a) Element of soil at failure (b) Mohr's circles

(c) Straight line approximation of curved failure envelope

Fig. 15.27 Shear strength of soil

While analysing the stability of an embankment, the designer uses one of the two methods in vogue. These are: (i) total stress method, and (ii) effective stress method. Laboratory tests (for determining shear strength parameters of a soil) for the total stress method are performed such that the experimental conditions with regard to pore pressure simulate the pore pressure conditions in the embankment at failure and the shear strength of the soil sample is measured in terms of total stress without pore pressure measurements. For the effective stress method, pore pressures on the potential failure surface are estimated and the shear strength is determined in terms of the effective stresses using Eq. (15.31). The total stress method is relatively simpler. But, the effective stress method is more fundamental in nature and, hence, is recommended (12) for the stability analysis of embankment dams.

Laboratory Tests for Shear Strength of Compacted Impervious Soils There are three types of laboratory tests which are commonly used for the determination of shear strength of compacted impervious soils. These methods differ in the method of consolidating the soil sample before the sample is failed in shear. These methods are as follows:

Undrained test,
Consolidated-undrained test, and
Drained test.

In the undrained test (also known as the 'quick', 'unconsolidated-undrained' or 'Q' test), drainage or dissipation of pore pressure is not allowed at any stage of the test. The relationship between the shear strength and normal pressure, obtained in terms of total stress, is used in the stability analysis of an embankment dam for 'during' and 'immediately after' construction condition. The soil samples are tested at a moisture content and density which would prevail during or immediately after construction. If the moisture content corresponds to saturation level, it will be found that all soil samples (saturated) have the same shear strength if no consolidation (i.e., drainage) is allowed (14).

In consolidated-undrained test (also known as 'consolidated-quick' or 'R' test) the sample is first allowed to consolidate (with full pore pressure dissipation) under a specified consolidation pressure, and is then failed in shear without permitting drainage. To obtain strength parameters in terms of effective stress (i.e., c' and ϕ'), the pore pressure should be measured. These values are used for effective stress method of stability analysis. If the test values are to be used for the total stress method of stability analysis, the sample should be tested (without pore pressure measurement) at a water content which is anticipated in the dam during the period being analysed. It should often be difficult to determine the water content which would prevail in the dam. Hence, it is usual practice to conduct the tests on saturated samples for total stress method of analysis. The results of these tests would be useful for analysing sudden drawdown condition of impervious zones of embankment and foundation (14). The test results would also be useful for analysing the upstream slope during a partial pool condition and the downstream slope during steady seepage (12).

Drained test (also known as the 'slow' test or 'S' test) permits drainage and complete dissipation of pore pressure at all stages of the test. The strength parameters are determined in terms of effective stress. The results of this test are to be used for freely-draining soils in which pore pressures do not develop (12).

Shear Strength of Pervious Soils

In embankment sections of clean sand and gravel, the pore pressures develop primarily due to seepage flow. The changes in pore pressure on account of changes in the embankment volume are short-lived and can be neglected. Consequently, the effective stress method of stability analysis is used for sections of pervious sand and gravel. The strength characteristics of sand and gravel are determined by drained tests. Because of the limitations of the size of test equipment, it may not be possible to test gravels of larger size. The strength characteristics of such soil can safely be assumed as those obtained for finer fractions of the same soil (4).

Factor of Safety

In most of the stability analysis methods for embankment dams, it is assumed that a slope might fail by a mass of soil sliding on a failure surface. At the moment of failure, the shear strength of the soil is fully mobilised on the entire failure surface and the overall slope as well as each of its parts are in the static equilibrium. For stable slopes, the shear strength mobilised under equilibrium conditions is less than the available shear strength and this is conventionally expressed in terms of a factor of safety, F defined as (14).

$$F = \frac{\text{shear strength available}}{\text{shear strength required for stability}} \qquad (15.32)$$

The acceptable value of factor of safety for different conditions of stability of an embankment dam depends on method of analysis, site conditions, size of the dam, functions of the reservoir, and so forth.

Methods of Stability Analysis for Embankment Dams

There are several methods for analysing the stability of embankment dams. Of these, the limit equilibrium methods are the ones most commonly used. In these methods, a number of failure surfaces are analysed to determine their factors of safety. The minimum of these values is taken as the factor of safety for the slope under consideration. The failure surface corresponding to the minimum factor of safety, F is termed the ''critical failure surface''. Obviously, for stable slopes, the value of F should be greater than unity. The methods of stability analysis recommended by Bureau of Indian standards are (12): (i) the standard method of slices, and

(ii) the wedge method.

Standard Method of Slices

This method (also known as the Swedish method of stability analysis) is the simplest method of stability analysis of embankment dams. It assumes that the forces acting on the sides of a slice do not affect the maximum shear strength which can develop on the bottom of the slice. This method of stability analysis was originally developed only for circular slip surfaces. However, it can be extended to non-circular slip surface also. The procedure for this method is as follows (4):

The trial sliding mass (i.e., the soil mass contained within the assumed failure sur- face) (Fig. 15.28) is divided into a number (usually 5 to 12) of slices which are usu- ally, but not necessarily, of equal width. The width is so chosen that the chord and arc subtended at the bottom of the slice are not much different in length and that the failure surface subtended by each slice passes through material of one type of soil.

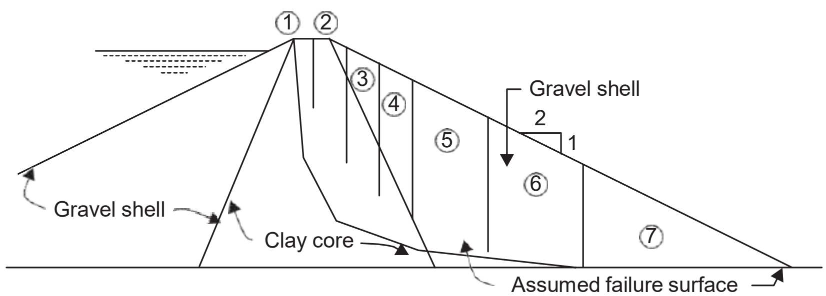


Fig. 15.28 Swedish method of stability analysis

For every slice of the trial sliding mass, the following forces are determined assum- ing the dam section to be of one unit length:

The total weight (W) of the slice which is equal to the area of the slice multiplied by suitable gross unit weight of the soil mass.

The force (U) due to pore pressure acting on the slice bottom is equal to the average unit pore pressure u, multiplied by the length of the bottom of the slice i.e., $U = ub/\cos\alpha$ where, b is the width of the slice and α is the angle between the vertical and normal drawn at the centre of the bottom of the slice under consid-eration.

The shear strength (C) due to cohesion for the slice under consideration is $c' b/\cos\alpha$ where, c' is the unit cohesion.

The normal and tangential components of the total weight W are N ($= W\cos\alpha$) and T ($= W\sin\alpha$).

The total shear force, i.e., shear strength S which develops on the bottom of the slice at failure equals $C + (N - U)\tan\phi'$. Here, ϕ' = angle of internal friction in terms of the effective stress.

In addition, there are intergranular forces (E) and forces due to pore pressure (U) acting on both sides of any given slice. While the magnitude and direction of

forces due to pore pressures can be estimated, the intergranular forces are not known. To make computational procedure simple, these forces (E_L and U_L) act- ing on one side of a given slice are assumed to be equal (in magnitude) and oppo-
site (in direction) to the forces acting on the other side of the slice (i.e., E_R and U_R). It should, however, be noted that ∑ ($E_L – E_R$) and ∑ ($U_L – U_R$) for the entire sliding mass are not zero.

The results of these computations are tabulated and the sums of the forces S and T are determined.

The factor of safety is computed from the relation

$$F = \frac{\sum S}{\sum T} = \frac{\sum [C + (N - U) \tan \phi']}{\sum T} \qquad (15.33)$$

By this method of analysis, one obtains a conservative value of the factor of safety. This is due to complete neglect of the intergranular forces and pore pressures acting on the sides of slices in the computations.

Alternatively the factor of safety for the chosen slip surface is computed using Taylor's ''Modified Swedish Method''. This method assumes that: (i) the directions of the intergranular forces acting on the sides of the slices are parallel to the average exterior slope of the embankment, and (ii) an equal proportion of the shear strength available is developed on the bottom of all the slices (4). The computational steps for Taylor's method applied to failuresurface of any arbitrary shape (Fig. 15.28) are as follows (4):

The trial sliding mass is divided into a suitable number of slice so that their chord length and arc length (subtended at the bottom of the slice) do not differ much andthe entire bottom of a slice is within one type of soil material.

For each slice [Fig. 15.29 (a)] following forces are computed:

The total weight W,

The forces due to pore pressure acting on the bottom and sides of the slices, i.e., U_L, U_R, and U_B, and

The cohesion force C acting on the bottom of the slice.

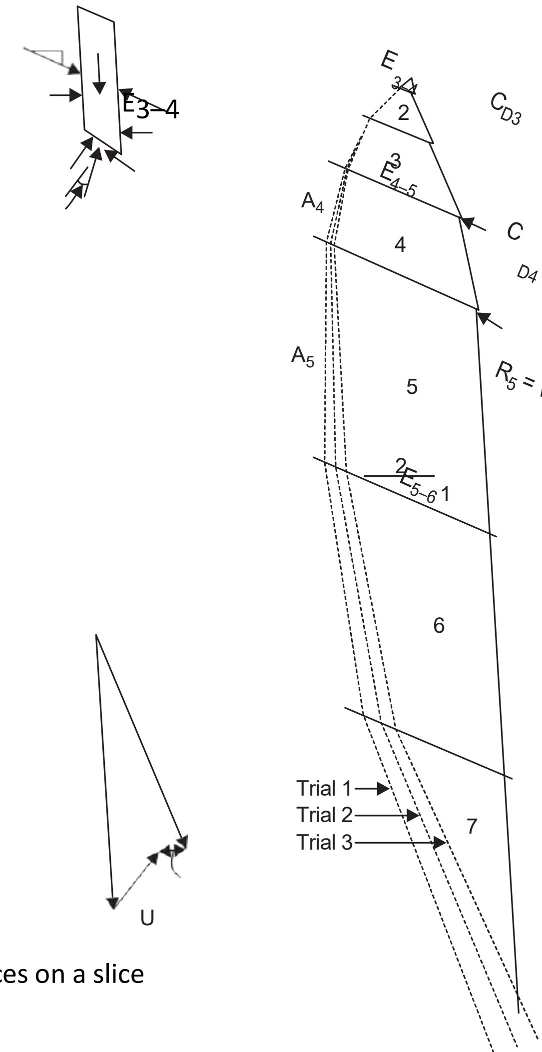
For each slice, the known forces W, U_L, U_R and U_B are resolved into a resultant R [Fig. 15.29 (b)].

The direction of the intergranular forces acting between the slices is assumed paral- lel to the average exterior slope of the embankment.

Assume a suitable value of factor of safety, say F_D, on the basis of stability analysis carried out by some approximate method, or otherwise, and determine

$$C_D = \frac{C}{F_D}$$

Draw composite force polygon [Fig. 15.29 (c)] for the whole trial sliding mass includ- ing all the forces acting on the individual slices.



Forces actingon slice 3

Resolution of knownforces on a slice

Composite forcepolygon

Fig. 15.29 Modified Swedish method of stability analysis

If the force polygon does not close, choose another value of F_D and compute C_D and redraw the composite force polygon. This is continued until one obtains the safety factor which closes the force polygon. This method can be similarly applied to two-wedge as well as three- wedge systems (Fig. 15.30). The modified Swedish method should be used for final stability analysis in all major embankment dams.

Wedge (or Sliding Block) Method

This method is used when the slip surface can be approximated by two or three straight lines. Such a situation arises when the slope is underlain by a strong stratum such as rock or there is a weak layer included within or beneath the slope. In such circumstances, an accurate stability analysis can be carried out by dividing the trial sliding mass into two or three blocks of soil and examining the equilibrium of each block. The upper block (or wedge) is called the driving (or active) block and the lower block is called the resisting (or passive) block. In a three-wedge system, the central block is called the sliding block (Fig. 15.30).

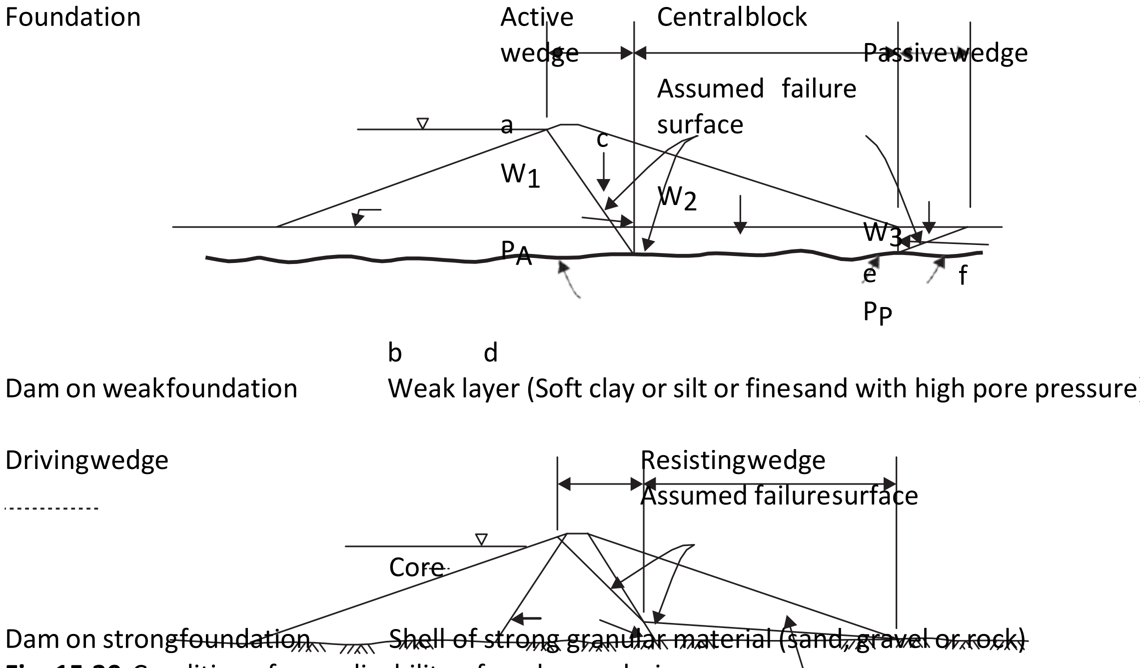


Fig. 15.30 Conditions for applicability of wedge analysis

The factor of safety can be estimated by any of the methods discussed earlier. Alternatively, assuming that the active and passive wedges are at failure and that the total forces on the vertical planes (bc and de) are horizontal, one can estimate the factor of safety as the ratio of the force P_1 available on bd to resist the movement of the central block and the unbalanced force, $P_A - P_P$. This means,

$$F = \frac{c_{bd} + (W_2 - U_{bd})\tan \phi_{bd}}{P_A - P_P} \qquad (15.34)$$

where, the subscript bd is used for the plane bd and the subscript 2 is for the central block.

Example 15.5 Determine the factor of safety for the slip surface shown in Fig. 15.31 for sudden drawdown condition with the following properties of the embankment material:

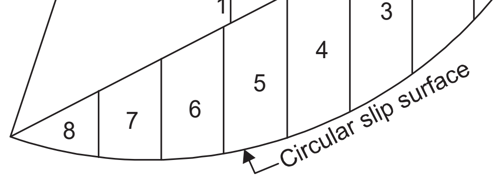


Fig. 15.31 Slip surface for Example 15.5

Saturated weight = 21.0 kN/m^3

Submerged weight = 11.0 kN/m^3

Cohesion = 24.5 kN/m^2

Angle of internal friction, ⍰⍰ = 35°

Angle ⍰, arc length, and area of different slices are given in the first four columns of Table 15.3.

Solution: Since pore pressures are not known, the driving force (T-component) and the resisting force (N-component) are calculated using saturated and submerged weights, respectively, and are shown in Table 15.3.

Table 15.3 Data and solution for Example 15.4

Slice	⍰ (degrees)	Arc length (metres)	Area of slice (m^2)	T-Component		N-Component	
				Weight, W (kN/m)	W sin ⍰ (kN/m)	Weight, W (kN/m)	W cos ⍰ (kN/m)
1	54.5	6.70	12.26	257.46	209.60	134.86	78.31
2	41	3.80	19.51	409.71	268.79	214.61	161.97
3	31	3.50	21.37	448.77	231.13	235.07	201.49
4	22	3.35	20.90	438.90	164.42	229.90	213.16
5	13	3.05	19.97	419.37	94.34	219.67	214.04
6	5	3.05	16.72	351.12	30.60	183.92	183.22
7	– 3.5	3.05	12.08	253.68	– 15.49	132.88	132.63
8	– 13	4.30	6.69	140.49	– 31.60	73.59	71.70
		30.80			951.79		1256.52

Using Eq. (15.33)

$$\text{Factor of Safety} = \frac{30.80 \text{ ⍰ } 24.5 \text{ ⍰ } 1256.52 \tan 35\text{⍰}}{951.79} = 1.72$$

Seismic Considerations in Stability Analysis

Seismic forces reduce the margin of safety of an embankment dam. Therefore, when an embankment dam is located in a seismic region, the stability analysis must also consider earthquake forces. During an earthquake, the ground surface oscillates randomly in different directions. This motion can be represented by horizontal and vertical components. A rigid structure is expected to follow the oscillations of its base in the absence of relative deformation from the base of the structure to its top. The amplitude of the oscillations and the acceleration vary along the height of the structure. An earth dam should be treated as a flexible structure for determining dynamic pressure due to earthquake. However, a simple method to account for earthquake forces in the design of structures is based on seismic coefficients. In this method, basic seismic coefficients or earthquake acceleration coefficients are used. The seismic coefficient is defined as the ratio of earthquake acceleration in a particular direction to the gravitational acceleration. If ⍰$_h$ is the horizontal earthquake acceleration coefficient then the additional inertial force of the soil mass (of the slice under consideration) is taken as ⍰$_h$W in the horizontal direction. Obviously, a force equal to ⍰$_h$W cos ⍰ is added to the tangential forces and ⍰$_h$W sin ⍰

is deducted from the forces acting in the normal direction. The factor of safety, therefore, becomes

$$F = \frac{\Sigma C + \Sigma (N - U - \alpha_h W \sin \theta) \tan \phi}{\Sigma (W \sin \theta + \alpha_h W \cos \theta)} \quad (15.35)$$

This simple way of accounting the seismic effects in the stability analysis is based on the pseudostatic concept in which the dynamic effects of an earthquake are replaced by a static force, and in which limit equilibrium is maintained (6).

Stability of Foundation

The factor of safety against shear for the foundation material is the ratio of the shear strength to shear stress at a location in the foundation where maximum intensity of shear stress occurs. Since the factor of safety corresponds to the maximum shear stress, its acceptable value should only be slightly greater than unity for the foundation of an embankment dam. The magnitude and location of the maximum shear stress can be determined approximately as follows:

The horizontal shear, S under a slope of the dam is equal to the difference between the lateral thrust on a vertical through the upper end of the slope and a vertical through the toe ofthe slope. Thus, with reference to Fig. 15.32,

$$S = \frac{1}{2} \rho g \left(h_1^2 - h_2^2\right) \tan^2 \left(45 - \frac{\phi_1}{2}\right) \quad (15.36)$$

where, ρ = average mass density of the soil,

g = acceleration due to gravity,

ϕ_1 = equivalent angle of internal friction, determined by the equation $\rho g h_1 \tan \phi_1 = c + \rho g h_1 \tan \phi$ where, c and ϕ are actual properties of the foundation soil, and h_1 and h_2 are heights, as shown in Fig. 15.32, of the upper and lower ends of the slope, respectively, above a stratum which is much stronger than the overlying foundation material.

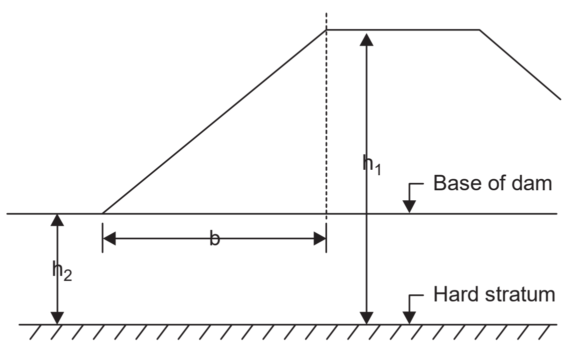


Fig. 15.32 Shear stress in foundation of embankment

The average shear stress s_a is equal to S/b. Here, b is the horizontal length of the slope. Results of photoelastic investigations indicate that the maximum intenstiy of shear equals 1.4 times the average shear s_a, and occurs at a distance of 0.4 b from the upper end of the slope. One can, alternatively, compare average values of shear stress and shear strength and accept a relatively higher factor of safety for the stability of the foundation of an embankment.

SLOPE PROTECTION

If the upstream slope of an earth dam, retaining a large reservoir, is composed of materials other than cobbles or rocks, it must be protected against damage by wave action. Based on past experience, the methods of upstream slope protection are: (i) dumped rock riprap, (ii) hand- placed stone pitching, (iii) monolithic RCC slab, and (iv) asphaltic concrete. Of these four methods, the dumped rock riprap overlying a finer filter layer (or layers) provides an excellent wave protection measure as it is least damaged by post-construction embankment settlement and is an effective dissipator of wave energy. A riprap layer should be designed such that: (i) the individual rocks are not moved out of place by the wave forces, and (ii) the filter underlying the riprap will not be washed out through the voids in the riprap layer. The filter itself should be able to prevent erosion of the underlying embankment material. The thickness and size of dumped rock riprap, as recommended by Bertram (15), are given in Table 15.4. Table 15.4 also gives the minimum thickness of underlying filter layer as recommended by the US Army Corps of Engineers.

Table 15.4 Thickness and size of dumped rock riprap

Maximum wave height (m)	Minimum average rock size D_{50} (cm)	Layer thickness (cm)	Minimum filter layer thickness (cm)
0-0.6	25	30	15
0.6-1.2	30	45	15
1.2-1.8	38	60	22.5
1.8-2.4	45	75	22.5
2.4-3.0	52	90	30

If the surface of the downstream slope of an earth dam consists of fine-grained soil, considerable erosion may be caused by either surface water runoff during rainstorms or wind during windstorms in arid regions. The erosion of the surface results in deep gullies (as deep as 3 m in severe cases) on the downstream slope at the abutment contacts as well as in the central portion of the dam.

A good cover of grass on the surface of the downstream slope holds the surface soil in place and provides most satisfactory and economical slope protection. However, in very dry areas, it may not be possible to spare enough water to grow and maintain the grass cover. In such situations, dumped rock riprap (used for upstream slope protection) can be used for the downstream slope as well.

INSTRUMENTATION

Instruments are installed in an embankment dam to measure pore pressures at different locations, and also the settlement and horizontal meovement of the dam. The measurements from these instruments enable the concerned authorities to compare the measured quantities with the corresponding values used by the designer. These measurements also provide a reliable basis for analysing the performance of the dam and for taking suitable steps to overcome a problem which may develop. Besides, these instruments provide valuable data for use in future designs of dams. The instruments installed in an embankment dam are piezometeres (for measuring pore water pressure), and instruments for measuring horizontal movements, foundation settlement, and embankment compressions. Instruments of the second type can be either internal instruments which are installed within the embankment during construction, or surface monuments consisting of concrete-embedded steel rods installed accuraterly along straight lines. The installed instruments should be simple in design with minimum moving parts, and should require no maintenance as most of the instruments would be embedded. The installation procedure should cause least interference in construction activities. The method of observation should also be simple.

EMBANKMENT CONSTRUCTION

The construction of an embankment mainly consists of the following activities:

Excavation of the material,

Hauling the excavated material to the site of the dam,

Mixing the material (either in the borrow pit or the embankment surface) with water to obtain the uniformity of desired water content and other properties,

Spreading the material in layers on the embankment surface, and

Compacting the spread material to the desired density.

Over the last fifty years there has been considerable development in larger and faster earth-handling equipment. The rate of embankment construction depends primarily on the amount and types of equipment used and can be of the order of 2000 m^3/day. The types of equipment generally used for the construction of embankment dams are as follows:

Excavating equipment, viz., power shovels, drag lines, scrapers, etc.,

Hauling equipment, viz., scrapers, truck, belt conveyors, etc.,

Spreading equipment, viz., bullozoers, graders, etc.,

Compacting equipment, viz., sheepsfoot and rubber-tyred rollers, etc., and

(vi) Watering equipment, viz., water trucks, hose, etc.

Depending upon the site conditions, some other types of equipment for specific purposes may also be needed.

GRAVITY DAMS

GENERAL

A gravity dam is a solid concrete or masonry structure which ensures stability against all applied loads by its weight alone without depending on arch or beam action. Such dams are usually straight in plan and approximately triangular in cross-section. Gravity dams are usually classified with reference to their structural height which is the difference in elevation between the top of the dam (i.e., the crown of the roadway, or the level of the walkway if there is no roadway) and the lowest point in the excavated foundation area, exclusive of such features as narrow fault zones (1). Gravity dams up to 100 ft (30.48 m) in height are generally considered as low dams. Dams of height between 100 ft (30.48 m) and 300 ft (91.44m) are designated as medium-height dams. Dams higher than 300 ft (91.44 m) are considered as high dams.

The downstream face of a gravity dam usually has a uniform slope which, if extended, would intersect the vertical upstream face at or near the maximum water level in the reservoir. The upper portion of the dam is made thick enough to accommodate the roadway or other required access as well as to resist the shock of floating objects in the reservoir. The upstream face of a gravity dam is usually kept vertical so that most of its weight is concentrated near the upstream face to resist effectively the tensile stresses due to the reservoir water loading. The thickness of the dam provides resistance to sliding and may, therefore, dictate the slope of the downstream face which is usually in the range of 0.7 to 0.8 (H) : 1(V). The thickness in the lower part of the dam may also be increased by an upstream batter.

When it is not feasible to locate the spillway in the abutment, it may be located on a portion of the dam in which case the section of the dam is modified at the top to accommodate the crest of the spillway and at the toe to accommodate the energy dissipator. The stability requirements of such overflow sections of gravity dams would be different from those of non-overflow gravity dams.

FORCES ON A GRAVITY DAM

The forces commonly included in the design of a gravity dam are shown in Fig. 16.1. These are as follows (2, 3, 4):

Dead Load

The dead load (W_c) includes the weight of concrete and the weight of appurtenances such as piers, gates, and bridges. All the dead load is assumed to be transmitted vertically to the foundation without transfer by shear between adjacent blocks.

Reservoir and Tail-water Loads (W_w , W_w', W_1, and W_1')

These are obtained from tail-water curves and range of water surface elevations in reservoir obtained from reservoir operation studies. These studies are based on operating and hydrologic data such as reservoir capacity, storage allocations, stream flow records, flood hydrographs, and reservoir releases for all purposes. In case of low overflow dams, the dynamic effect of the velocity of approach may be significant and should, therefore, be considered. If gates or other control features are used on the crest, they are treated as part of the dam so far as the application of water pressure is concerned. In case of non-overflow gravity dams, the tail-water should be adjusted for any retrogression. Any increase in tail-water pressure due to curvature of flow in the downstream bucket of an overflow type gravity dam should also be considered in the design of gravity dams (4).

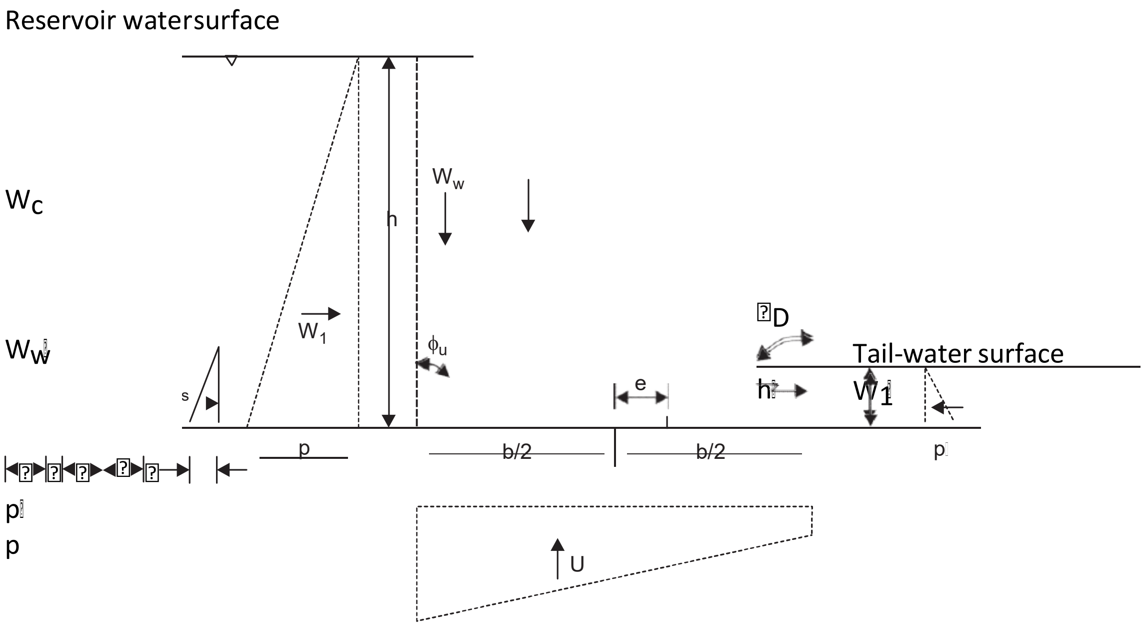


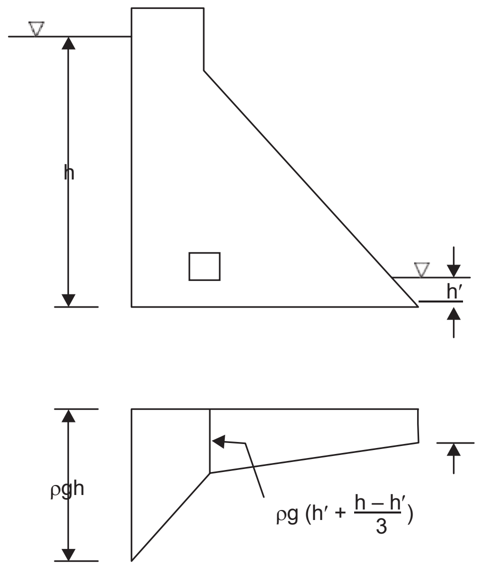
Fig. 16.1 Usual loading combination for a gravity dam

Uplift Forces

Uplift forces (U) occur due to internal hydraulic pressures in pores, cracks, and seams within the body of a dam, at the contact between the dam and its foundation, and within the foundation.The distribution of internal hydrostatic pressure along a horizontal section through a gravity dam is assumed to vary linearly from full reservoir pressure at the upstream face to zero or tail-water pressure at the downstream face, and to act over the entire area of the section. The pressure distribution is also adjusted depending upon the size, location, and spacing of internal drains. Experimental and analytical studies indicate that the drains set in from the upstream face at 5 per cent of the maximum reservoir depth and spaced laterally twice that distance will reduce the average pressure at the drains to approximately tail-water pressure plus one-third the difference between reservoir water and tail-water pressures (3) (Fig. 16.2). It is assumed that uplift forces are not affected by earthquakes (2).

Silt Load

The construction of a dam across a river carrying sediment invariably results in reservoir sedimentation which causes an additional force (W_S) on the upstream face of the dam. The horizontal silt pressure is assumed equivalent to a hydrostatic load exerted by a fluid with a mass density of 1360 kg/m^3. The vertical silt pressure is assumed equivalent to that exerted by a soil with a wet density of 1925 kg/m^3.



ρgh

Fig. 16.2 Modification in uplift force due to drain

Ice Pressure

If the designer anticipates the formation of an ice sheet of appreciable thickness and its remaining on the reservoir water surface for a long duration, the ice pressures must be computed using a suitable method of their estimation. In the absence of such a method, ice pressure may be taken as 250 kPa (250 kN/m^2) applied over the anticipated area of contact of ice with the face of the dam (2).

Wave Pressure

The upper portion of a dam is also subjected to the impact of waves. Wave pressure against massive dams of large height is usually of little importance. Wave pressure is related to wave height h_w as follows (2):

The maximum wave pressure p_w (in kilopascals) occurs at 0.125 h_w above the still water level and is given by the equation

$$p_w = 24\, h_w \quad (16.1)$$

where, h_w is the height of the wave in metres.

The total wave force P_w (in kilonewtons) is given by

$$P_w = 20\, h_w^2 \quad (16.2)$$

and acts at 0.375 h_w above the still water level in the downstream direction.

The wave height h_w can be calculated using the following relations:

$$h_w = 0.032 \sqrt{VF} + 0.76 - 0.27\, F^{1/4} \quad \text{for } F < 32 \text{ km} \quad (16.3)$$

$$h_w = 0.032 \sqrt{VF} \quad \text{for } F > 32 \text{ km} \quad (16.4)$$

Here, V is the wind velocity in kilometres per hour and F is the fetch in kilometres.

The height of the wave and the wind set-up decide the freeboard which is the vertical distance between the top of the dam and the still water level. The wind set-up S (in metres) isestimated by the Zuider Zee formula

$$S = \frac{V^2 F}{62{,}000\, D} \qquad (16.5)$$

in which, D is the average depth in metres over the fetch distance F.

The minimum freeboard should be equal to wind set-up plus $\frac{4}{3}$ times wave height above the normal pool elevation or above maximum reservoir level corresponding to the design flood, whichever gives higher crest elevation for the dam (2). The freeboard shall not, however, be less than 1.0 m above the mean water level corresponding to the design flood.

Earthquake

Gravity dams are elastic structures which may be excited to resonate by seismic disturbances.Such dams should be designed so that they remain elastic when subjected to the design earthquake. The design earthquake should be determined considering (i) historical records ofearthquakes to obtain frequency of occurrence versus magnitude, (ii) useful life of the dam, and (iii) statistical approach to determine probable occurrence of earthquakes of various magnitudes during the life of the dam. A gravity dam should also be designed to withstand the maximum credible earthquake which is defined as the one having a magnitude usually largerthan any historical recorded earthquake (3).

Earthquakes impart random oscillations to the dam which increase the water and silt pressures acting on the dam and also the stresses within the dam. An earthquake movement may take place in any direction. Both horizontal and vertical eathquake loads should be applied in the direction which produces the most unfavourable conditions. For a gravity dam, when the reservoir is full, the most unfavourable direction of earthquake movement is upstream (so that the inertial forces acting downstream may result in resultant force intersecting the base of the dam outside middle-third of the base besides increasing the water load and, therefore, the increased overturning moment) and is downward for vertical earthquake movement as it causes the concrete, and water above the sloping faces of the dam to weigh less resulting in reduced stability of the dam. When the reservoir is empty, more unfavourable is the downstream ground motion causing inertial forces to act upstream so that the resultant may intersect the base of the dam outside middle-third of the base. The effect of earthquake forces depends on (i) their magnitude which, in turn, depends on the severity of the earthquake, (ii) the mass of thestructure and its elasticity, and (iii) the earthquake effects on the water load. For estimation of earthquake load, knowledge of earthquake acceleration or intensity, usually expressed in relation to acceleration due to gravity g, is useful. This ratio of earthquake acceleration to gravitational acceleration is termed seismic coefficient and is designated as α_h. The value of seismic coefficient for horizontal as well as vertical earthquake accelerations for different zones of the country are different and can be obtained from the Codes. Considering a structure of mass M moving with an acceleration α_h g in the horizontal direction during an earthquake,the horizontal earthquake force acting on the structure, P_e is given as

$$P = M \alpha_h g = \frac{W}{g} \alpha_h g = \alpha_h W$$

e g

where, W is the weight of the structure. The value of ?$_h$ has usually been taken as 0.1 in the absence of any other specified value. Similarly, the value of the seismic coefficient in the vertical
direction can be taken as 0.05.

The inertia of water in the reservoir also produces a force on the face of the dam during an earthquake. For dams with vertical or sloping upstream face, the variation of horizontal hydrodynamic earthquake pressure with depth is given by the following equations (5):

$$p_e = c_1 \alpha_h \rho g h \qquad (16.6)$$

and

$$c_1 = \frac{c_m}{2}\left[\frac{y}{h}\left(2 - \frac{y}{h}\right) + \sqrt{\frac{y}{h}\left(2 - \frac{y}{h}\right)}\right] \qquad (16.7)$$

where, p_e = hydrodynamic earthquake pressure normal to the face,

c_1 = a dimensionless pressure coefficient,

α_h = ratio of horizontal acceleration due to earthquake and the gravitational acceleration, i.e., horizontal acceleration factor,

ρ = mass density of water,

g = acceleration due to gravity.

h = depth of reservoir,

y = vertical distance from the reservoir surface to the elevation under consideration, and c_m = the maximum value of c_1 for a given slope (Fig. 16.3).

90°

80°

70°

60°

50°

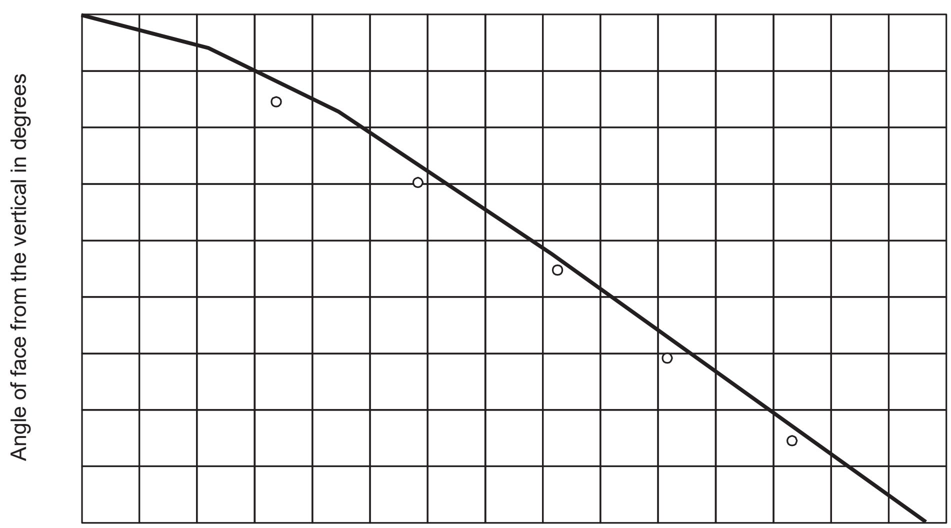
40°

30°

20°

10°

0°



0 0.1 0.2 0.3 0.4 0.5 0.6 0.7

Maximum pressure coefficient c_m

Fig. 16.3 Variation of c_m with inclination of the upstream face

Let V_{pe} (or V_{pe}') represent change in horizontal component of reservoir (or tail-water) load on the face above a section due to horizontal earthquake loads and it is computed for each increment of elevation selected for the study and the totals obtained by summation because of the nonlinear response (1). Likewise, M_{pe} (or M_{pe}') which represents the moment of V_{pe} (or V_{pe}') about the centre of gravity of the section, is computed. The inertia forces for concrete in the dam should be computed for each increment of height, using the average acceleration factor for that increment. The inertia forces to be used while considering an elevation in the dam are the summation of all the incremental forces above that elevation and the total of their moments about the centre of gravity at the elevation being considered. The horizontal concrete inertia force (V_e) and its moment (M_e) can be calculated using Simpson's rule (1). Alternatively, V_{pe} and M_{pe} can be obtained from the following equations (4):

$V_{pe} = 0.726\ p_e\ y$

and $M_{pe} = 0.299\ p_e\ y^2$

The effects of vertical accelerations may be determined using the appropriate forces, moments, and the vertical acceleration factor α_v. The forces and moments due to water pressure normal to the faces of the dam and those due to the dead loads should be multiplied by the appropriate acceleration factors to determine the increase (or decrease) caused by the vertical downward (or upward) accelerations. The effect of earthquake on uplift forces is considered negligible.

Dams having upstream face as a combination of vertical and sloping faces are analysed as follows (4):

If the height of the vertical portion of the upstream face of a dam is equal to or greater than one-half the total height of the dam, analyse the dam as if it has a vertical upstream face throughout.

If the height of the vertical portion of the upstream face of a dam is less than half the total height of the dam, use the pressure which would occur if the upstream face has a constant slope (equal to the slope of the sloping portion of the upstream face) from the water surface elevation to the heel of the dam.

Other Miscellaneous Loads

In addition to the above-mentioned forces there may be thermal loads and vertical water loading too. If the contraction joints are grouted, the horizontal thrusts, caused by volumetric increases due to rising temperature, will produce load transfer across joints. This load transfer increases the twist effects and the loads at the abutments (3). Vertical water loading is exerted by the weight of the water on sloping upstream and downstream faces of the dam. The vertical component of the water flowing over the spillway is not included in the analysis as water tends to attain the spouting velocity which reduces pressure on the dam. Any negative pressure which may develop on the spillway crest is also neglected. However, any sub-atmospheric pressure developing on the downstream sloping surface of the spillway due to lack of aeration should be considered by treating them as positive load (acting in the downstream direction) applied on the upstream face.

The design of gravity dams must consider most adverse combination of probable load conditions (1). Combinations of loads whose simultaneous occurrence is highly improbable may, however, be excluded. Most load combinations can be categorised as usual, unusual, or extreme. For example, normal design reservoir elevation with appropriate dead loads, uplift, silt, ice, tail-water, and thermal loads corresponding to usual temperature make a typical usual type load combination. Unusual load combination considers maximum design reservoir elevation with the loads of usual type load combination. Extreme load combination results when the effects of maximum credible earthquake are included in the usual load combination.

CAUSES OF FAILURE OF A GRAVITY DAM

A gravity dam may fail on account of overturning. For a gravity dam to be safe against overturning, the dimensions of the dam should be such that the resultant of all the forces intersects the base of the dam within its middle-third portion. Consider any horizontal section (including the base) of a gravity dam and the resultant of all the forces acting on the dam above the section. If the line of action of this resultant passes outside the downstream edge of the section, the dam would overturn. However, the section of a gravity dam is such that the line of action of the resultant force is within the upstream and downstream edges of the section and overturning would never results. But, if the line of action of the resultant passes sufficiently outside the middle-third of the horizontal section, it may cause crushing of the downstream edge of the section. This would reduce the effective width and, hence, the sliding resistance of the section and may cause the resultant to pass outside the dam section. Further, when the resultant passes downstream of the middle-third of the horizontal section, it induces tensile stresses at the upstream edge of the section. These tensile stresses may cause cracks in the dam section which would result in increased uplift pressure. The stabilising forces would, thus, be reduced. It follows, therefore, that before a gravity dam overturns bodily, other types of failures, such as crushing of toe material, sliding, cracking of the material due to tension, and increase in uplift may occur. A gravity dam is considered safe against overturning if the criteria of: (i) no tension on the upstream face, (ii) adequate resistance against sliding, and (iii) suitable quality and sufficient strength of concrete/masonry of dam and its foundation aresatisfied.

Concrete and masonry are relatively weak in tension and as such the design of a gravity dam should ensure that there are no tensile stresses anywhere in the dam section. In very high gravity dams, however, if it becomes difficult to ensure such a condition, one may allow small tensile stresses not exceeding 50 N/cm^2 under the most adverse condition of loading.

The horizontal forces acting on a dam above any horizontal plane may cause failure of the dam due to sliding if these driving forces are more than the resistance to sliding on the plane. The resistance to sliding is due to the frictional resistance and shearing strength of the material along the plane under consideration. The shear-friction factor of safety, F_S, which is a measure of stability against sliding or shearing, can be expressed as follows:

$$F_s = \frac{CA + \mu \Sigma W}{\Sigma H} \qquad (16.8)$$

where, C = unit cohesion,

A = area of the plane considered (A can be replaced by the width of the plane, if one considers unit length of the dam),

ΣW = sum of all vertical forces acting on the plane,

μ = coefficient of internal friction, and

ΣH = sum of driving shear forces i.e., resultant horizontal forces.

The shear-friction factor of safety can be used to determine the stability against sliding or shearing at any horizontal section within a dam, its contact with the foundation or through the foundation along any plane of weakness. The minimum allowable values of F_S for gravity dam are 3.0, 2.0, and 1.0 for the usual, unusual, and extreme loading combinations, respectively (3). The value of F_S for any plane of weakness within the foundation should not be less than 4.0, 2.7, and 1.3 for the usual, unusual, and extreme loading combinations, respectively (3).

Generally, the acceptable factor of safety against overturning and shear for normal or usual loading condition is taken as 2.0. The corresponding value for extreme loading condition is 1.25. The acceptable value of the sliding factor (= the ratio of the sum of the horizontal forces and the sum of the vertical forces) is the ratio of the coefficient of static friction and the chosen factor of safety.

The maximum allowable compressive stress for concrete in a gravity dam should be less than the specified compressive strength of the concrete divided by 3.0, 2.0, and 1.0 for usual, unusual, and extreme load combinations, respectively. The compressive stress should not exceed 1035 N/cm^2 and 1550 N/cm^2 for usual and unusual load combinations, respectively (3).

The maximum allowable compressive stress in the foundation should be less than the compressive strength of the foundation divided by 4.0, 2.7, and 1.3 for usual, unusual, and extreme load combinations, respectively. These values of factor of safety are higher than those for concrete so as to provide for uncertainties in estimating the foundation properties.

STRESS ANALYSIS OF GRAVITY DAMS

Stress analyses of gravity dams can be carried out using one of the following methods depending upon the configuration of the dam, continuity between the blocks, and the degree of refinement required.

The gravity method,

The trial-load method, and

The finite element method

The gravity method of analysis is used for designing straight gravity dams in which the transverse contraction joints of a gravity dam are neither keyed nor grouted. When the transverse contraction joints of a gravity dam are keyed, irrespective of their grouting, it becomes a three-dimensional problem and one should use the trial load method. This method assumes that the dam consists of three systems, viz., the vertical cantilevers, the horizontal beams, and the twisted elements. Each of these systems is assumed to occupy the entire volume of the structure and is independent of the others. The loads on the dam are divided among these systems in such a manner as to cause equal deflections and rotations at conjugate points (1). This is achieved by trial process. The gravity method may, however, be used for a preliminary analysis of keyed and grouted dams. The finite element method, developed in recent years, can be used for both two-dimensional as well as three-dimensional problems. In this book, only the gravity method has been discussed.

Gravity Method

The gravity method of stress analysis is applicable to the general case of a gravity section when its blocks are not made monolithic by keying and grouting the joints between them. All these blocks of the gravity section act independently, and the load is transmitted to the foundation by cantilever action, and is resisted by the weight of the cantilever. The following assumptions are made in the gravity method of analysis (1):

Concrete in the dam is a homogeneous, isotropic, and uniformly elastic material.

No differential movements occur at the site of the dam due to the water loads on walls and base of the reservoir.

All loads are transmitted to the foundation by the gravity action of vertical and par- allel cantilevers which receive no support from the adjacent cantilever elements oneither side.

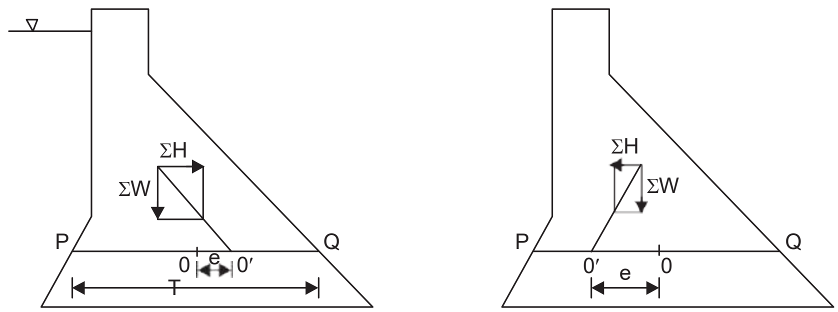
Normal stresses on horizontal planes vary linearly from the upstream face to down- stream face.

Horizontal shear stresses have a parabolic variation across horizontal planes from the upstream face to downstream face of the dam.

The assumptions at serial numbers (iv) and (v) above are substantially correct, except for horizontal planes near the base of the dam where the effects of foundation yielding affect the stress distributions in the dam. Such effects are, however, usually small in dams of low or medium height. But, these effects may be significant in high dams in which cases stresses near the base should be checked by other suitable methods of stress analysis.

As shown in Fig. 16.4, ΣW and ΣH represent, respectively, the sum of all the resultant vertical and horizontal forces acting on a horizontal plane (represented by the section PQ) of a gravity dam. The resultant R of ΣW and ΣH intersects the section PQ at O′ while O represents the centroid of the plane under consideration. The distance between O and O′ is called the eccentricity of loading, e. When e is not equal to zero, the loading on the plane is eccentric and the normal stress σ_{yx} at any point (on the section PQ) x away from the centroid O is given as

$$\sigma_{yx} = \frac{\Sigma W}{A} \pm \frac{(\Sigma W)\, e}{I} x \qquad (16.9)$$



Reservoir full condition (b) Reservoir empty condition

Fig. 16.4 Resultant force on a gravity dam

Here, A represents the area of the plane PQ, and I is the moment of inertia of the plane PQ about an axis passing through its centroid and parallel to the length of the dam. It should be noted that whereas the direct stress (= $\Sigma W/A$) at every point of the section PQ is always compressive, the nature of the bending stress (= $(\Sigma W)\, e\, x/I$) depends on the location of O′ with respect to O. If O′ lies between O and Q, there will be compressive bending stress for any point between O and Q, and tensile bending stress for any point between O and P. Accordingly, when the reservoir is full, one should use the positive sign in Eq. (16.9) for all points between O and Q, and the negative sign for all points between O and P. Similarly, when the reservoir is empty (in which case ΣH may be an earthquake force acting in the upstream direction), and O′ lies between O and P, one should use the positive sign for all points between O and P, and the negative sign for all points between O and Q.

Considering unit length of the dam and the horizontal distance between the upstream edge P and the downstream edge Q of the plane PQ as T, one can write A = T and I = $T^3/12$. Thus, Eq. (16.9) reduces to

$$\sigma_{yx} = \frac{\Sigma W}{T}\left(1 \pm \frac{12ex}{T^2}\right) \qquad (16.10)$$

One can use this equation for determining the normal stress on the base of the dam BB′ (Fig. 16.5) also. If the width of the base BB′ is b, Eq. (16.10) for the base of the dam reduces to

$$\square_{yx} = \frac{\square W}{b}\left(1 \square \frac{12ex}{b^2}\right) \qquad (16.11)$$

For the toe (B′) and also the heel (B) of the dam, x = b/2. Hence, the normal stresses at the toe (σ_{yD}) as well as the heel (σ_{yU}) of the dam are as follows:

When the reservoir is full,

$$\sigma_{yD} = \frac{\Sigma W}{b}\left(1 + \frac{6e}{b}\right) \quad (16.12)$$

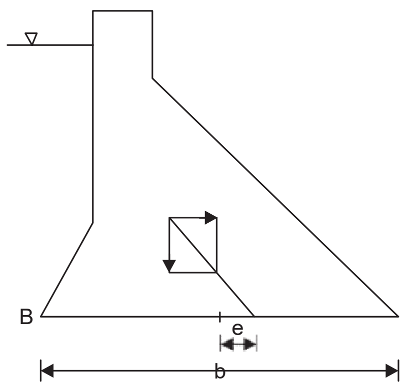
$$\sigma_{yU} = \frac{\Sigma W}{b}\left(1 - \frac{6e}{b}\right) \quad (16.13)$$

When the reservoir is empty,

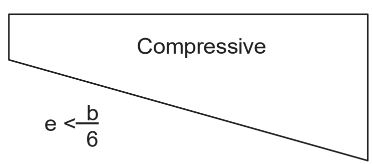
$$\sigma_{yD} = \frac{\Sigma W}{b}\left(1 - \frac{6e}{b}\right) \quad (16.14)$$

$$\sigma_{yU} = \frac{\Sigma W}{b}\left(1 + \frac{6e}{b}\right) \quad (16.15)$$

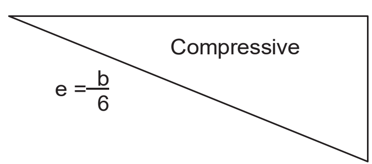
B′



$< \frac{2\,\Sigma W}{b}$



$\frac{2\,\Sigma W}{b}$



$> \frac{2\,\Sigma W}{b}$

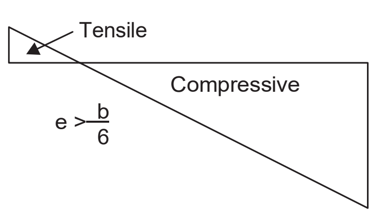


Fig. 16.5 Normal stresses on the base of a gravity dam

These equations indicate that if e is less than or equal to b/6, the stress is compressive all along the base and when e is greater than b/6 there can be tensile stresses on the base. The stress distributions for different values of e, when the reservoir is full, have been shown in Fig. 16.5. This means that if there has to be no tension at any point of the base of the dam, the resultant for all conditions of loading must meet the base within the middle-third of the base.

The principal planes and principal stresses enable one to know the range of the stresses acting at a point and thus design the structure on the basis of extreme values. A plane on which only normal stresses act is known as a principal plane. Shear stresses are not present on such a plane. Accordingly, the upstream and downstream faces of a gravity dam, having tail- water, are principal planes as the only force acting on these surfaces is on account of water pressure which acts normal to these surfaces. Further, at any point in a structure the principal planes are mutually perpendicular. Therefore, other principal planes would be at right angles to the upstream and downstream faces of a gravity dam. In an infinitesimal triangular element PQR at the toe of a gravity dam, (Fig. 16.6), the plane QR is at right angle to the downstream face, PQ. Hence, PQ and QR are the principal planes, and PR is part of the base of the dam. The stresses acting on the principal planes PQ and QR are, respectively, p' (tail-water pressure)
and σ_{1D}, as shown in Fig. 16.6, and are the principal stresses. The normal and tangential stresses acting on PR are σ_{yD} and $(\tau_{yx})_D$, respectively. Since the element is very small, the stresses can be considered to be acting at a point. Considering the equilibrium of the element
PQR, the algebraic sum of all the forces in the vertical direction should be zero. If one considers the unit length of the dam, then

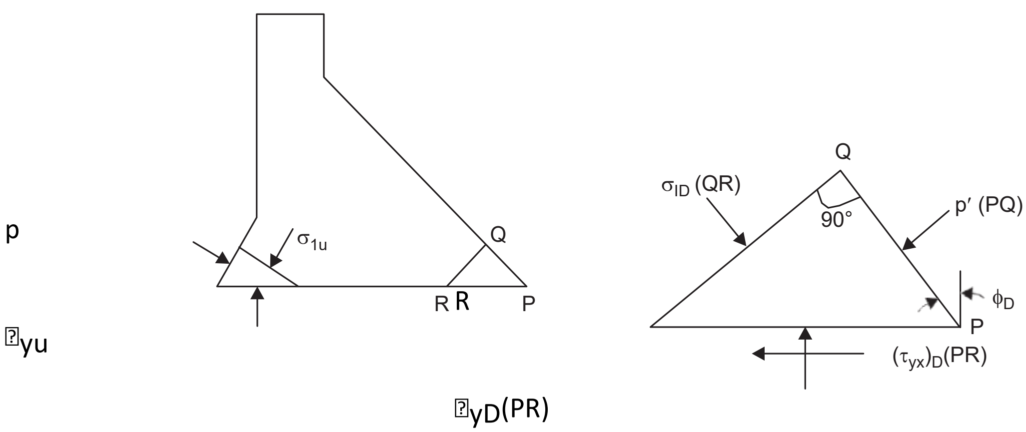


Fig. 16.6 Principal stresses in a gravity dam

σ_{1D} (QR) cos ϕ_D + p' (PQ) sin ϕ_D – σ_{yD} (PR) = 0

or σ_{1D} (PR) $\cos^2 \phi_D$ + p' (PR) $\sin^2 \phi_D$ – σ_{yD} (PR) = 0

$\therefore$ $\sigma_{1D} = \sigma_{yD} \sec^2 \phi_D - p' \tan^2 \phi_D$ (16.16) Thus, knowing p' and σ_{yD} [from Eq. (16.11)] one can obtain, from Eq. (16.16), the principal stress σ_{1D} at the toe of the dam. Usually p' is either zero (no tail-water) or very small
in comparison to σ_{1D}. Therefore, σ_{1D} is the major principal stress and p' is the minor principal

stress. When p is zero, Eq. (16.16) reduces to

$$_{1D} = _{yD} \sec^2 _D \quad (16.17)$$

Considering the hydrodynamic pressure p_e due to earthquake acceleration (towards the reservoir), the effective minor principal stress becomes p – p_e and Eq. (16.16) becomes

$$_{1D} = _{yD} \sec^2 _D - (p - p_e) \tan^2 _D \quad (16.18)$$

When there is no tail-water, both p⍰ and p_e⍰ are zero, and Eq. (16.17) is used for the calculation of $⍰_{1D}$.

Similarly, considering an infinitesimal element at the heel of the dam (Fig. 16.6), one can obtain expression for $⍰_{1U}$ as follows:

$⍰_{1U} = ⍰_{yU} \sec^2 ⍰_U - (p + p_e) \tan^2 ⍰_U$ (16.19) For the condition of empty reservoir, p = p_e = 0 and, hence,

$⍰_{1U} = ⍰_{yU} \sec^2 ⍰_U$ (16.20)

When the reservoir is full, the intensity of water pressure p is usually higher than the normal stress $⍰_{1U}$. Therefore, at the heel, p is the major principal stress and $⍰_{1U}$ is the minor principal stress. For vertical upstream face, $⍰_U$ = 0 and, therefore, $⍰_{1U}$ equals $⍰_{yU}$.

Again, resolving the forces acting on the infinitesimal element PQR in the horizontal direction and equating their algebraic sum to zero for the equilibrium condition, one gets

$(⍰_{yx})_D$ (PR) + p⍰ (PQ) cos $⍰_D$ – $⍰_{1D}$ (QR) sin $⍰_D$ = 0

which yields $(⍰_{yx})_D = (⍰_{1D} - p⍰) \sin ⍰_D \cos ⍰_D$

$= (⍰_{yD} \sec^2 ⍰_D - p⍰ \tan^2 ⍰_D - p⍰) \sin ⍰_D \cos ⍰_D$

⍰ $(⍰_{yx})_D = (⍰_{yD} - p⍰) \tan ⍰_D$ (16.21) Similarly, considering the equilibrium of the element at the heel of the dam,

$(⍰_{yx})_U = -(⍰_{yU} - p) \tan ⍰_U$ (16.22)

Including the effects of earthquake acceleration, Eqs. (16.21) and (16.22) reduce to $(⍰_{yx})_D$ = $[⍰_{yD} - (p⍰ - p_e⍰)] \tan ⍰_D$ (16.23)

and $(⍰_{yx})_U = -[⍰_{yU} - (p + p_e)] \tan ⍰_U$ (16.24)

In the same way, one can calculate the principal and shear stresses at the upstream and downstream faces of the dam at any horizontal section by considering only the forces acting above the section.

ELEMENTARY PROFILE OF A GRAVITY DAM

The stability conditions required to be met for a gravity dam, subjected only to its self-weight W, force due to water pressure P, and uplift force U can be satisfied by a simple right-angled triangular section (Fig. 16.7) with its apex at the reservoir water level, and which is adequately wide at the base where the water pressure is maximum. Such a section is said to be an elementary profile of a gravity dam. For the empty-reservoir condition the only force acting on the dam is its self-weight whose line of action will meet the base at b/3 from the heel of the dam and thus satisfy the stability requirement of no tension. The base width of the elementary profile is determined for satisfying no tension and no sliding criteria as given below, and the higher of the two base widths is chosen for the elementary profile.

For the elementary profile shown in Fig. 16.7, if one considers that the resultant R of all the three forces W_c (= 0.5 s ⍰gbh), W_1 (= 0.5 ⍰gh^2), and U (= 0.5 ⍰ghbc⍰) (here, s = specific gravity of concrete and c⍰ is a correction factor for uplift force) passes through the downstream middle-third point, one gets

$$(0.5\ s\ ⍰gbh) \frac{b}{3}\ ⍰\ (0.5\ ⍰gh^2) \frac{h}{3}\ ⍰\ (0.5\ ⍰ghbc⍰) \frac{b}{3} = 0$$

or $b^2 (s - c) = h^2$

W1

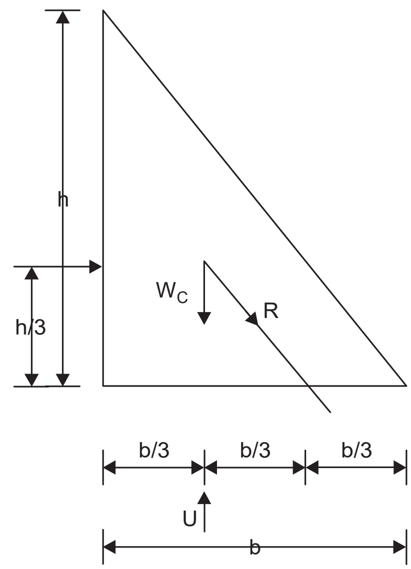


Fig. 16.7 Elementary profile of a gravity dam

or $b = \frac{h}{\sqrt{s - c'}}$ (16.25)

For $c' = 1$, $b = \frac{h}{\sqrt{s - 1}}$ (16.26)

and if uplift is ignored, $c' = 0$

□ $b = \frac{h}{\sqrt{s}}$ (16.27)

For no-sliding requirement, one obtains □ $(W_C - U) = P = W_1$ in which, □ is shear-friction coefficient.

or □ (0.5 s□gbh – 0.5 □ghbc□) = 0.5 □gh^2

or $b = \frac{h}{□ (s □ c□)}$ (16.28)

For c = 1 , $b = \frac{h}{□ (s □ 1)}$ (16.29)

and for no uplift, c = 0, and $b = \frac{h}{□s}$ (16.30)

It is obvious that for satisfying the requirement of stability, the elementary profile of a gravity dam should have minimum base width equal to the higher of the base widths obtained from no-sliding and no-tension criteria.

Again, for an elementary profile, □W = $(W_C - U)$

or W = $\frac{1}{2}$ b□gh (s □ c□) Σ —

$$\therefore \quad \sigma_{yx} = \frac{\Sigma W}{b}\left(1 + \frac{12ex}{b^2}\right) \tag{16.31}$$

For no tension in the dam, e = b/6

Therefore, at the toe of the dam (i.e., x = b/2)

$$\sigma_{yD} = \frac{2\Sigma W}{b}$$

$$\sigma_{yD} = \rho g h (s - c') \tag{16.32}$$

and at the heel of the dam (i.e., x = – b/2)

$\sigma_{yU} = 0$ Accordingly, the principal stress $\sigma_{1D} = \sigma_{yD} \sec^2 \phi_D$

$$\therefore \quad \sigma_{1D} = \rho g h (s - c') [1 + (b/h)^2]$$

$$= \rho g h (s - c') \left[1 + \frac{1}{(s - c')}\right]$$

$$= \rho g h (s - c' + 1) \tag{16.33}$$

Similarly, $(\tau_{yx})_D = \sigma_{yD} \tan \phi_D = \rho g h (s - c') \dfrac{b}{h}$

$$= \rho g h (s - c') \frac{1}{\sqrt{s - c'}}$$

$$= \rho g h \sqrt{s - c'} \tag{16.34}$$

The principal and shear stresses at the heel are, obviously, zero.

Similarly, when the reservoir is empty, $\Sigma W = 0.5\, \rho g b h s$

$\sigma_{yD} = 0$

$$\sigma_{1U} = \sigma_{yU} = \frac{2\Sigma W}{b} = \rho g h s \tag{16.35}$$

Sometimes, depending upon whether or not the compressive stress at the toe σ_{1D} exceeds the maximum permissible stress σ_m for the material of the dam, a gravity dam is called a ‘high’ or ‘low’ dam. On this basis, the limiting height h_l is obtained by equating the expression for σ_{1D} with σ_m. Thus,

$$\sigma_m = \rho g h_l (s - c' + 1)$$

or
$$h_l = \frac{\sigma_m}{\rho g (s - c' + 1)} \tag{16.36}$$

If the height of a gravity dam is less than h_l, it is a low dam; otherwise, it is a high dam.

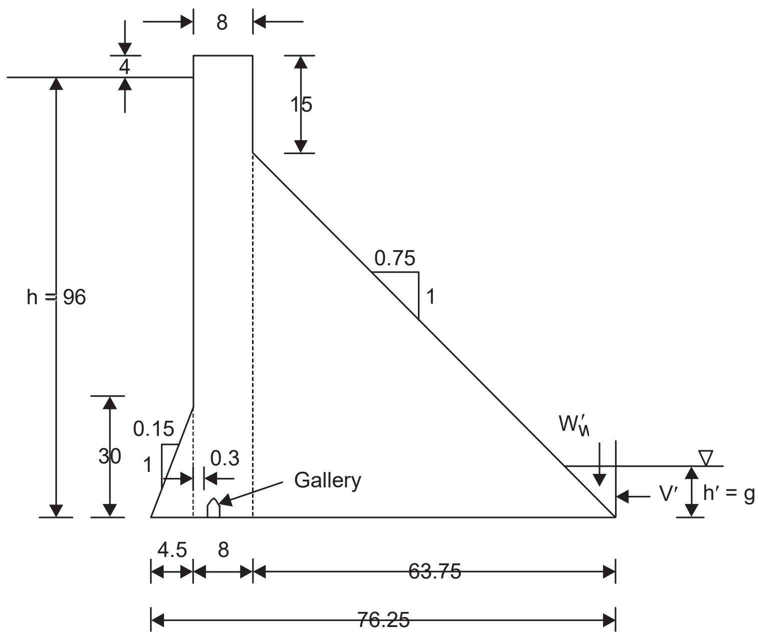
DESIGN OF A GRAVITY DAM

An elementary profile is only an ideal profile which needs to be modified for adoption in actual practice. Modifications would include provision of a finite crest width, suitable freeboard, batter in the lower part of the upstream face, and a flatter downstream face. The design of a gravity dam involves assuming its tentative profile and then dividing it into a number of zones by horizontal planes for stability analysis at the level of each dividing horizontal plane.

The analysis can be either two-dimensional or three-dimensional. The following example illustrates the two-dimensional method of analysis of gravity dams.

Example 16.1 For the profile of a gravity dam shown in Fig. 16.8, compute principal stresses for usual loading and vertical stresses for extreme loading at the heel and toe of the base of the dam. Also determine factors of safety against overturning and sliding as well as shear-friction factors of safety for usual loading and extreme loading (with drains inoperative) conditions. Consider only downward earthquake acceleration for extreme loading condition. Sediment is deposited to a height of 15 m in the reservoir. Other data are as follows: Coefficient of shear friction, ⍰ = 0.7 (usual loading)

= 0.85 (extreme loading) Shear strength at concrete-rock contact, C = 150 × 10^4 N/m^2 Weight density of concrete = 2.4 × 10^4 N/m^3



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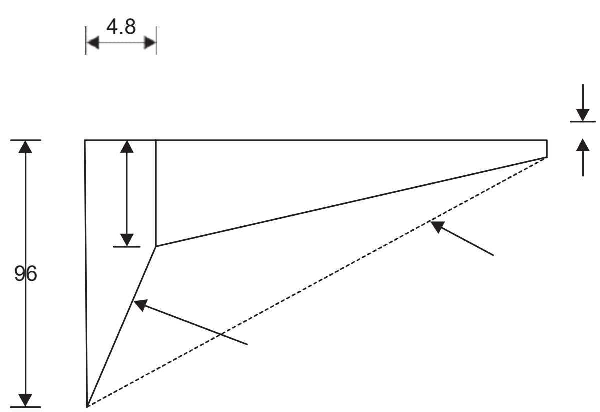

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Drains inoperative

Drains operative

Uplift pressure head diagram

Fig. 16.8 Profile and uplift pressure diagram for the gravity dam of Example 16.1

Weight density of water = 1×10^4 N/m^3

$\alpha_h = 0.1$; $\alpha_v = 0.05$

Solution:

Computation of Stresses:

Usual loading combination (normal design reservoir elevation with appropriate dead loads, uplift (with drains operative), silt, ice, tail-water, and thermal loads corre- sponding to usual temperature):

Resultant vertical force = ΣW = sum of vertical forces at sl. nos. 1, 2 (i), 3 (i), and 4(ii)of Table 16.1.

$= (8584.50 + 394.88 - 2000.68 + 32.48) \times 10^4$

$= 7011.18 \times 10^4$ N

Resultant horizontal force = ΣH = sum of horizontal forces at sl. nos. 2 (ii) and 4 (i) of Table 16.1.

$= (-4567.50 - 153.00) \times 10^4$

$= -4720.50 \times 10^4$ N

Moment about toe of the dam at the base = ΣM = sum of moments at sl. nos. 1,2, 3 (i),and 4 of Table 16.1.

$= [418302.75 + 27091.99 - 147334.50 - 96183.57 + 1662.88] \times 10^4$

$= 203539.55 \times 10^4$ Nm

Distance of the resultant from the toe, $y = \dfrac{\Sigma M}{\Sigma W} = \dfrac{203539.55 \times 10^4}{7011.18 \times 10^4} = 29.03$ m

$\therefore$ Eccentricity, $e = 38.125 - 29.03 = 9.10$ m

(The resultant passes through the downstream of the centre of the base). Using Eqs. (16.12) and (16.13)

$$\sigma_{yD} = \frac{\Sigma W}{b}\left(1 + \frac{6e}{b}\right) = \frac{7011.18 \times 10^4}{76.25}\left(1 + \frac{6 \times 9.10}{76.25}\right)$$

$= 157.79 \times 10^4$ N/m^2

$$\sigma_{yU} = \frac{\Sigma W}{b}\left(1 - \frac{6e}{b}\right) = \frac{7011.18 \times 10^4}{76.25}\left(1 - \frac{6 \times 9.10}{76.25}\right)$$

$= 26.11 \times 10^4$ N/m^2

Using Eq. (16.16), the major principal stress at the toe,

$\sigma_{1D} = \sigma_{yD} \sec^2 \phi_D - p' \tan^2 \phi_D$

$\therefore \quad \sigma_{1D} = 157.79 \times 10^4 \times 1.5625 - 9 \times 10^4 \times 0.5625$

$= 239.66 \times 10^4$ N/m^2

Using Eq. (16.21), shear stress at the toe, $(\tau_{yx})_D = (\sigma_{yD} - p') \tan \phi_D$

$\therefore \quad (\tau_{yx})_D = (157.79 - 9) \times 10^4 \times 0.75$

$= 111.59 \times 10^4$ N/m^2

Table 16.1 Computation of forces and moments for unit length of dam section

Sl. No.	Type of load	Force computations (10^4 newtons)	Magnitude of forces: Vertical forces (downward + ve) (10^4 newtons)	Magnitude of forces: Horizontal forces (upstream + ve) (10^4 newtons)	Moment arm (metres)	Moment about the toe (anticlock-wise + ve) (10^4 Nm)
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1.	Dead load W_c	1 × 30 × 4.5 × 0.5 × 2.4	+ 162.00		73.25	+ 11866.50
		1 × 100 × 8 × 2.4	+ 1920.00		67.75	+ 130080.00
		1 × 85 × 63.75 × 0.5 × 2.4	+ 6502.50		42.50	+ 276356.25
			+ 8584.50			+ 418302.75
2.	Water load					
(*i*)	Vertical					
(*a*)	Head-water W_w	1 × 66 × 4.5 × 1.0	+ 297.00		76.25 – 2.25 = 74.00	+21978.00
		1 × 0.5 × 30 × 4.5 × 1.0	+ 67.50		76.25 – 1.50 = 74.75	+5045.63
(*b*)	Tail-water W_w'	1 × 0.5 × 9.0 × 6.75 × 1.0	+ 30.38		(9 × 0.75)/3 = 2.25	+ 68.36
			+ 394.88			+ 27091.99
(*ii*)	Horizontal					
(*a*)	Head-water W_l	1 × 0.5 × 96 × 96 × 1.0		– 4908.00	32.00	–147456.00
(*b*)	Tail-water W_l'	1 × 0.5 × 9 × 9 × 1.0		+ 40.50	3.00	+ 121.50
				– 4567.50		– 147334.50
3.	Uplift force, U					
(*i*)	Drains operative	1 × 38 × 4.8 × 1	– 182.40		73.85	– 13470.24
		1 × 58 × 4.8 × 0.5 × 1	– 139.20		74.65	– 10391.28
		1 × 29 × 71.45 × 0.5 × 1	– 1036.03		47.63	– 49345.87
		1 × 9 × 71.45 × 1	– 643.05		35.73	– 22976.18
			– 2000.68			– 96183.57
(*ii*)	Drains inoperative	1 × 0.5 × 87 × 76.25 × 1	– 3316.88		50.83	– 168597.01
		1 × 9 × 76.25 × 1	– 686.25		38.13	– 26166.71
			– 4003.13			– 194763.72

(Contd.)...

(1)	(2)	(3)	(4)	(5)	(6)	(7)
4.	Load due to sediment deposit W_s					
(i)	Excess horizontal pressure	1 × 0.5 × 15 × 15 × 1.36		– 153	15/3 = 5.00	– 765.00
(ii)	Excess vertical load	1 × 0.5 × 15 × 2.25 × 1.925	32.48		76.25 – 4.5/3 = 74.75	+ 2427.88
						+ 1662.88
5.	Earthquake forces					
(i)	Inertial horizontal force due to weight of the dam	162.00 × 0.1		– 16.20	10.00	– 162.00
		1920.00 × 0.1		– 192.00	50.00	– 9600.00
		6502.50 × 0.1		– 650.25	28.33	– 18421.58
				– 858.45		– 28183.58
(ii)	Hydrodynamic force	At the base $c = c_m = 0.73$ (for $\phi_U = 0$)				
(a)	Head-water ($c_1 = c_m$ for $y = h$)	$V_{pe} = 0.726\ (0.73 \times 0.1 \times 1 \times 96) \times 96$		– 488.43		
		$M_{pe} = 0.299\ (0.73 \times 0.1 \times 1 \times 96) \times (96)^2$				– 19311.13
(b)	Tail-water[1]	At the base $c = c_m = 0.47$ (for $\phi_D = \tan^{-1}(0.75)$)				
		$V_{pe} = 0.726\ (0.47 \times 0.1 \times 1 \times 9) \times 9$		– 2.76		
		$M_{pe} = 0.299\ (0.47 \times 0.1 \times 1 \times 9) \times (9)^2$				– 10.25
				– 491.19		19321.38

[1]**Hydrodynamic force due to tail -water has been considered negative for the most critical condition.**

Using Eq. (16.19), with $p_e = 0$, the minor principal stress at the heel

$\sigma_{1U} = \sigma_{yU} \sec^2 \phi_U - p' \tan^2 \phi_U$

$= 26.11 \times 10^4 \times 1.0225 - 96 \times 10^4 \times 0.0225$

$= 24.54 \times 10^4$ N/m^2

Using Eq (16.22), shear stress at the heel, $(\tau_{yx})_U = (\sigma_{yU} - p') \tan \phi_U$

$\therefore \quad (\tau_{yx})_U = -(26.11 - 96.00) \times 10^4 \times 0.15$

$= 10.48 \times 10^4$ N/m^2

Further, major principal stress at the heel = p = 96×10^4 N/m^2 and minor principal stress at the toe = $p' = 9.0 \times 10^4$ N/m^2

$$\text{Factor of safety for overturning} = \frac{\text{stabilising moment}}{\text{overturning moment}}$$

$$= \frac{(418302.75 + 27091.99 + 1662.88) \times 10^4}{(147334.50 + 96183.57) \times 10^4} = 1.84$$

$$\text{Sliding factor} = \frac{\Sigma H}{\Sigma W}$$

$$= \frac{4720.55 \times 10^4}{7011.18 \times 10^4} = 0.67$$

Shear-friction factor of safety (with drains operative),

$$F_s = \frac{Cb \times 1 + \mu \Sigma W}{\Sigma H}$$

$$= \frac{150 \times 10^4 \times 76.25 \times 1 + 0.7(7011.18 \times 10^4)}{4720.50 \times 10^4} = 3.46$$

Extreme loading combination (usual loading combination with drains inoperative and the loading due to earthquake):

The inertial and hydrodynamic forces and corresponding moments due to horizontal earthquake have been computed as shown in Table 16.1. The effect of vertical earthquake can be included in stability computations by multiplying the forces by $(1 + \alpha_v)$ and $(1 - \alpha_v)$ for upward and downward accelerations, respectively. Since the computation of hydrodynamic force involves the use of unit weight of water, the hydrodynamic force will also be modified by vertical acceleration due to earthquake. Further, the effect of earthquake on uplift forces is considered negligible. For 'reservoir full' condition, the downward earthquake acceleration results in more critical condition. Therefore, the following computations have been worked out for the downward earthquake acceleration only.

Resultant vertical force with downward earthquake acceleration

$= (8584.50 + 394.88 + 32.48) \times 10^4 \times 0.95 - 2000.68 \times 10^4$

$= 6560.59 \times 10^4$ N

Resultant horizontal force with downward earthquake acceleration

$= (4567.50 + 153 + 858.45 + 491.19) \times 10^4$

$= 6070.14 \times 10^4$ N

Resultant moment about the toe with downward acceleration

$= (418302.75 + 27091.99 - 147334.50 + 1662.88 - 28183.58 - 19321.38) \times 10^4 \times 0.95 - 194763.72 \times 10^4$

$= 44843.53 \times 10^4$ Nm

Now, $\quad y = \dfrac{\Sigma M}{\Sigma W} = \dfrac{44843.53 \times 10^4}{6560.59 \times 10^4} = 6.835$ m

$\therefore$ Eccentricity, $e = 38.125 - 6.835 = 31.29$ m

The resultant passes through the downstream side of the centre of the base. The value of e is more than b/6 i.e., 12.71 m. Therefore, there would be tensile stresses around the heel of the dam. The vertical stresses at the toe and heel with downward earthquake acceleration are,

$$\sigma_{yD} = \frac{\Sigma W}{b}\left(1 + \frac{6e}{b}\right) = \frac{6560.59 \times 10^4}{76.25}\left(1 + \frac{6 \times 31.29}{76.25}\right)$$

$$= 297.89 \times 10^4 \text{ N/m}^2$$

and

$$\sigma_{yU} = \frac{\Sigma W}{b}\left(1 - \frac{6e}{b}\right) = \frac{6560.59 \times 10^4}{76.25}\left(1 - \frac{6 \times 31.29}{76.25}\right)$$

$$= -125.81 \times 10^4 \text{ N/m}^2$$

Factor of safety against overturning

$$= \frac{(418302.75 + 27091.99 + 1662.88) \times 10^4}{(147334.50 + 194763.72 + 28183.58 + 19321.38) \times 10^4} = 1.15$$

Sliding factor $= \dfrac{\Sigma H}{\Sigma W}$

$$= \frac{(4567.50 + 153 + 858.45 + 491.19) \times 10^4}{6560.59 \times 10^4}$$

$$= \frac{6070.14}{6560.59} = 0.925$$

Shear-friction factor of safety, $F_S = \dfrac{Cb \times 1 + \mu \Sigma W}{\Sigma H}$

$$= \frac{150 \times 10^4 \times 76.25 \times 1 + 0.7 \times 6560.59 \times 10^4}{6070.14 \times 10^4}$$

$$= \frac{16029.91}{6070.14} = 2.64$$

FOUNDATION TREATMENT

The foundation of a gravity dam should be firm and free of major faults which, if present, may require costly foundation treatment. The entire loose overburden over the area of the foundationto be occupied by the base of the dam should be removed. The dam itself must be based on the firm material which can withstand the loads imposed by the dam, reservoir, and other appurtenant structures. To consolidate the rock foundation and to make it an effective barrier to seepage under the dam, the foundation is often grouted. Grouting consists of filling the cracks and voids in the foundation with grout mixtures (cement-water mixtures) under pressure. The spacing, length, pattern of grout holes, and grouting procedure depend on the height of the structure and the geologic characteristics of the foundation. Grouting operations are carried out from the surface of the excavated foundation or from galleries within the dam or from tunnels driven into the abutments or from other suitable locations, such as the upstream filletof the dam.

For the purpose of seepage control, a deep grout curtain is constructed near the heel of the dam by drilling deep holes and grouting them under high pressures. These holes, if drilled from a gallery, are identified as 'A' holes. Curtain grouting is carried out only after consolidation grouting so that the higher grouting pressure does not cause displacement in the rock or loss of grout through surface cracks. In low dams, galleries are not provided and high-pressure grouting is carried out through curtain holes located in the upstream fillet of the dam before reservoir filling begins. Such grouting holes are identified as 'C' holes.

Consolidation grouting to fill voids, fracture zones, and cracks at and below the surface of the excavated foundation is accomplished by drilling and grouting relatively shallow holes. These holes are identified as 'B' holes and the grouting is carried out at low pressures.

Water-cement ratios for grout mixes depend on the permeability of the rock foundationand may be around 6 : 1 for consolidation grouting. Pressures for consolidation grouting depend on the strength characteristics of the foundation and may vary over wide range of 70 to 700 kPa. Holes of diameters of approximately 5 cm, spaced at about 5 m intervals, are usually drilled to a depth of about 15 m depending upon the local conditions.

Even after providing a grout curtain some water will percolate through and around the grout curtain. This water, if not removed, may build up very high hydrostatic pressures at the base of the structure. Hence, this water must be suitably drained. This is achieved by drilling one or more lines of holes downstream of the grout curtain. The spacing, depth, diameter, and pattern of these holes would depend on the foundation conditions. Drain holes are drilled only after completion of all foundation grouting. These can be drilled from foundation and drainage galleries within the dam, or from the downstream face of the dam. A suitable system for the collection and safe disposal of the drainage water must also be provided.

MASS CONCRETE FOR DAMS

Mass concrete can be defined (6) as any large volume of cast-in-place concrete with dimensions large enough to require measures to cope with the generation of heat and attendant volume change to minimize cracking. Like regular concrete, mass concrete too is primarily composedof cement, aggregate, and water. Additionally, it has pozzolans and other admixtures to improve its characteristics.

Proper proportioning of mass concrete mixture is aimed at: (i) achieving economy, (ii) low temperature-rise potential with adequate workability for placing, and (iii) adequate strength, durability, and impermeability to serve efficiently the structure in which it is used. For this purpose, "low heat" portland cement would always be preferred for massive structures such as dams. Obviously, both economy and low rise in temperature would be achieved by limiting the cement content of mass concrete to as low a value as possible.

Aggregate grading has considerable effect on the workability of concrete. Fine aggregate is defined (6) as aggregate passing No. 4 (4.76 mm) sieve. It may be composed of natural grains, manufactured grains obtained by crushing larger size rock particles, or a mixture of the two. Fine aggregate should consist of hard, dense, durable, and uncoated rock fragments, and should not contain harmful grains of clay, silt, dust, mica, organic matter or other impurities to such an extent that they affect adversely the desired properties of concrete.

Coarse aggregate is defined (6) as gravel, crushed gravel, or a crushed rock, or a mixture of these, generally within the range of 4.76 mm to 150 mm in size. Coarse aggregate should also consist of hard, dense, durable, and uncoated rock fragments. Rock which is very fragile or which tends to degrade during processing, transporting, or in storage should be avoided. Further, rocks having an absorption greater than 3 per cent or a specific gravity less than 2.5are not considered suitable for mass concrete.

The shape of the aggregate particles affects workability and, hence, water requirement. Round particles provide best workability. More than 25 per cent of flat (width-thickness ratio greater than 3) and elongated (length-width ratio greater than 3) particles should not be permitted in each size group.

Water used for preparing mass concrete mix should neither significantly affect the hydration reaction of portland cement nor interfere with the phenomena that are intended to occur during the mixing, placing, and curing of concrete. Water which is suitable for human consumption is acceptable for use in mass concrete.

Pozzolans are used to improve the workability and quality of concrete, to effect economy, and to protect against disruptive expansion caused by the reaction between different constituents of mass concrete. A pozzolan is defined (6) as a siliceous or siliceous and aluminous material which, in itself, possesses little or no cementitious value but will, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties. Natural pozzolanic materials occur in the form of obsidian, pumicite, volcanic ashes, tuffs, clays, shales, and diatomaceous earth. Most of these pozzolans require grinding. Fly ash (fuel dust from power plants burning coal) too can be an excellent pozzolan as it has a low carbon content, a fineness about the same as that of portland cement, and occurs in the form of very fine glassy spheres.

Admixtures are generally used to alter the properties of concrete (such as increased workability or reduced water content, acceleration or retardation of setting time, acceleration of strength development, and improved resistance to weather and chemical attacks) to make it more suitable for a particular purpose. For example, calcium chloride can be used to accelerate strength development in mass concrete during winter. Air-entraining admixtures (inexpensive soaps, detergents, etc.) entrain air which greatly improves the workability of concrete and thus permits the use of harsher and more poorly graded aggregates and also those of undesirable shapes.

Mass concrete usually contains low portions of cementing materials, sand and water. Uniformity of batching is, therefore, very essential for achieving the desired level of workability. Since mass concrete is produced on a large scale, the uniformity of batching is easily attained by using the most effective methods and equipment such as (6): (i) finish screening of coarse aggregate at the batching plant, preferably on horizontal screens, (ii) a device for instant reading of approximate moisture content of sand, (iii) refinements in batching equipment such as full- scale springless dials which register all stages of the weighing operation, (iv) automatic weighing and cutoff features, (v) interlocks to prevent recharging when some material remains in a scale hopper, and (vi) graphic recording of the various weighing and mixing operations.

Mass concrete is best placed in successive layers which should not exceed 45 to 50 cm in thickness with 10 to 15 cm maximum-size aggregate and less than 4 cm slump placed with 3 to

6 m^3 buckets and powerful 15 cm diameter vibrators. Vibration is vital for successful use of efficient, lean, low-slump mass concrete. Mass concrete is best cured with water (for the additional cooling in warm weather) for at least 14 days or up to 28 days if pozzolan is used as one of the cementing materials.

A major problem associated with mass concrete is the probability of high tensile stresses due to generation of heat by the hydration of cement along with subsequent differential cooling. A decrease in temperature of concrete causes volumetric changes resulting in the development of tensile stresses and consequent cracking in the concrete mass. Such cracking in concrete dams is undesirable as it adversely affects their water-tightness, internal stresses, durability, and appearance. Temperature drop is, therefore, controlled by controlling placing temperature, limiting the temperature-rise potential of concrete, controlling lift thickness and placing schedule, and removal of heat through embedded cooling coils. From the considerations of temperature control, the longer interval between successive lifts is preferred provided the ambient temperatures are lower than those of the concrete surfaces while internal temperature is rising. Presently, it is common practice to pre-cool mass concrete before its placement. Best uniformity of mix is attained when maximum use of ice is made for precooling mass concrete.

STRUCTURAL JOINTS

Joints in concrete dams are essentially designed cracks which are suitably located and treated to minimise undesirable effects. These joints are of three types.

Contraction joints prevent tensile cracks on account of volumetric shrinkage due to drop in temperature. A concrete dam is usually constructed in blocks separated by the transverse contraction joints which are normal to the axis of the dam and are continuous from the upstream face to the downstream face. Longitudinal contraction joints in the blocks formed by the transverse contraction joints are also sometimes considered necessary. The contraction joints are vertical and normally extend from the foundation to the top of the dam. Reinforcement should not extend across a contraction joint.

Expansion joints accommodate volumetric increase due to rise in temperature besides preventing transfer of stress between different units of the structure. Contraction and expansion joints are constructed in such a manner that there is no bond between the adjacent units of the structure.

Construction joint is the surface of the previously placed concrete upon or against which new concrete is to be placed. Besides permitting subsequent placing of concrete, these joints facilitate construction, reduce shrinkage stresses, and permit installation and embedded metal work. Suitable measures are adopted to ensure proper bond between the previously placed concrete and new concrete. These measures include cleaning by high-velocity water jets and roughening the surface of the previously placed concrete. A thin mortar layer is sometimes placed on the surface before placing new concrete. Construction joints do not require water seals and keys.

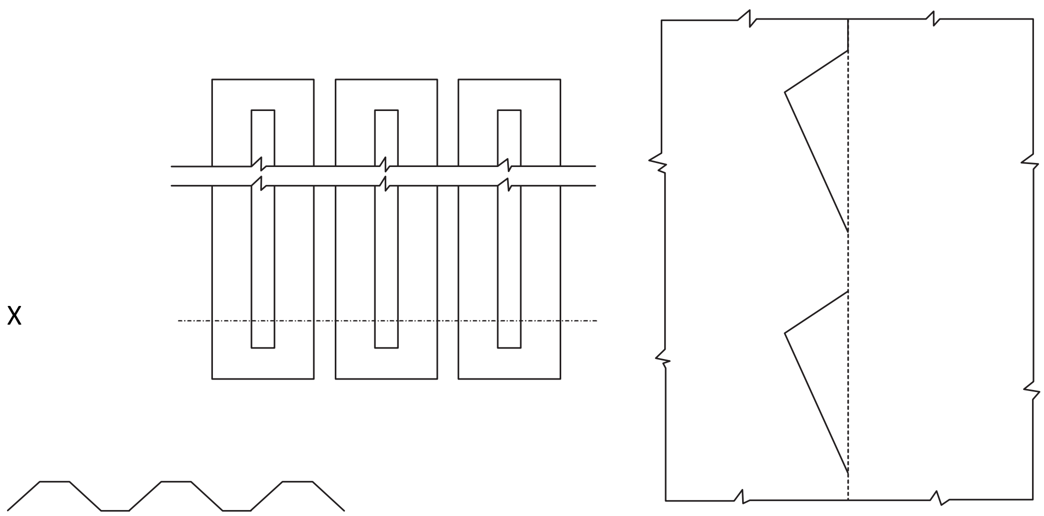
KEYS AND WATER SEALS

Vertical keys in transverse joints [Fig. 16.9 (a)] and horizontal keys in longitudinal joints [Fig. 16.9 (b)] are provided in a dam to increase the shearing resistance between its adjacent concrete blocks. The resulting structure has better stability due to the transfer of load from one block to another through the keys. The keys also increase the percolation distance through the joints and thus reduce water leakage. They also hasten the sealing of the joints with sediment deposits.

Shear keys provided in longitudinal contraction joints improve the stability of the dam by increasing the resistance to vertical shear.

Contraction joints are sometimes grouted to bind the blocks together so that the structure behaves like a monolithic mass. Even if the stability of the dam does not require the entire mass to act as a monolith, the longitudinal contraction joints must always be grouted so thatblocks in a transverse row act monolithically.

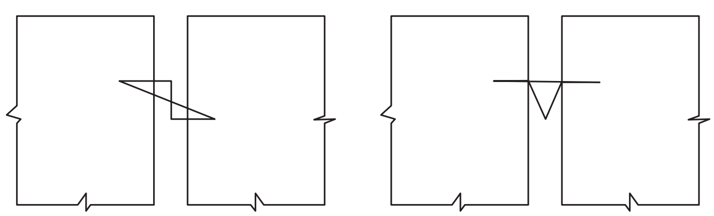
The opening of the transverse joints between adjacent blocks creates a passage for leakage of water from the reservoir to the downstream face of a dam. To prevent this leakage, seals (Fig. 16.10) are installed in the joints adjacent to the upstream face. Seals in longitudinal and transverse joints are also useful during grouting operations for confining the grout mixture to the joint. The most common type of seal used in concrete dams has been a metal seal which is embedded in concrete across the joint. In addition, polyvinyl chloride (PVC) seals and rubberseals have also been used.



Section XX

Vertical keys fortransverse joints   Horizontal keys forlongitudinal joints

Fig. 16.9 Typical keys for joints in gravity dam



Z-type   (b) M-type

Fig. 16.10 Metallic water stops in transverse joints

GALLERIES

A gallery is an opening within a dam that provides access into or through the dam. These may run either longitudinally or transversely and may be either horizontal or inclined. The following are the common types and uses of galleries (1):

Drainage galleries provide a drainage way for water percolating through the up- stream face or seeping through the foundation.

Grouting galleries provide space for drilling and grouting the foundation.

Inspection galleries provide access to the interior of the structure for observing its behaviour after completion.

Gate galleries (or chambers or vaults) provide access to, and room for, such mechani- cal and electrical equipment as are used for the operation of gates in spillways and outlet works.

Cable galleries provide access through the dam for control cables and/or power ca- bles and related equipment.

Visitors' galleries provide access routes for visitors.

Other galleries may be needed in a particular dam to meet special requirements, such as the artificial cooling of concrete blocks, the grouting of contraction joints, and so on.

INSTRUMENTATION

The behaviour of concrete gravity dam as well as its foundation is observed through suitable instruments during the periods of construction, reservoir filling, and operation of the reservoir. Instruments are employed for measurements of strain, temperature, stress, deflection, and deformation of the foundation. The data obtained from various instruments are helpful not only for assessing the safety of the structure during construction as well as operation periods, but, also for future design and operational studies.

The instruments (or the methods) employed for obtaining information on the behaviour of a concrete dam and its foundation can be broadly classified into two categories. The first category includes such instruments as are either embedded in the concrete mass of the structure or placed on surface of the dam and its appurtenant structures. Such instruments measure strain, stress, contraction joint opening, temperature, concrete pore pressures, and foundation or deformation. The second category of instruments include precise surveying instruments which make measurements of horizontal and/or vertical deformation using targets located on the downstream face or top of a dam, inside the galleries and vertical wells in a dam, in tunnels, and on the abutments.

OUTLETS

An outlet in a dam is a combination of structures and equipment required for the safe operation and control of water released from a reservoir to serve different objectives. Outlets are classifiedaccording to the purpose they serve. River outlets regulate flows to the river and control the water level in the reservoir. Besides, the river outlets may be useful for increasing the flow downstream of the dam alongwith the normal spillway discharge. In addition, river outlets may also act as a flood control regulator to release waters temporarily stored in flood control storage space or to evacuate storage in anticipation of flood inflows. Further, river outlets may also serve to empty the reservoir for inspection, repair, and maintenance of the upstream face of the dam and other structures which are normally inundated.

Irrigation, municipal water supply, and industrial outlets control the flow of water into a canal, pipeline, or river to satisfy specified needs. The design of these outlets will depend primarily on irrigation, commercial, industrial, and residential water needs and also on the capacity requirements with the reservoir at a predetermined elevation as well as the amount of control required as the elevation of the reservoir fluctuates.

Power outlets provide passage of water to the turbines for generation of hydropower. The power outlets should be so designed as to minimise hydraulic losses and to obtain the maximum economy in construction as well as operation.

SPILLWAYS

GENERAL

The occurrence of a flood in an unobstructed natural stream is considered to be a natural event for which no individual or group is held responsible. However, if a flood occurs on account of the failure of an artificial obstruction (such as a dam) constructed across a natural stream, the agency responsible for the construction of the obstruction is held responsible. Embankment dams constructed of earth or rockfill material are very likely to be destroyed, if overtopped. Concrete dams may, however, tolerate moderate overtopping. The damage to life and property on account of the failure of a dam would be catastrophic. As such, there must always be a provision to release excess water safely when the reservoir has been filled to its capacity so that the dam itself is not overtopped. This is achieved by constructing a spillway. Spillways release safely the surplus water which cannot be contained in the reservoir created by the dam. The surplus water is usually drawn from the top of the reservoir and conveyed through an artificial waterway back to the river downstream of the dam or to some other natural drainage channel. Spillway can be constructed either as part of the main dam, such as in overflow section of a concrete dam or as a separate structure altogether. Besides being capable of releasing surplus water, a spillway must be able to meet hydraulic and structural requirements and must be located such that spillway discharges do not damage the toe of the dam. Insufficient spillway capacity and/or the failure of a spillway will cause widerspread damage and loss of life. As such, the design criteria for a spillway are usually conservative. The inflow design flood, used to determine the spillway capacity, is also estimated conservatively.

The frequency of flow over a spillway would mainly depend on the runoff characteristics of the drainage area, reservoir storage, and the available outlet and/or diversion capacity. For example, at a dam with storage and outlet capacities relatively small (compared to normal river flows), spillway will be used almost continuously. Under favourable site conditions, one should examine the possibility of providing an auxiliary spillway in conjunction with a smaller service spillway. In such a situation, the service spillway is designed to pass frequent floods of smaller magnitude. The auxiliary spillway operates only when flood of magnitude larger than the discharging capacity of the service spillway is passing. Sometimes, the capacity of outlet structures may be increased so that these may also serve as service spillways. The auxiliary spillway is used infrequently and, hence, is not designed for the same degree of safety as required for other spillways.

At some projects, emergency spillways are also provided for additional safety to meet emergencies not anticipated in the normal design. Such emergencies could be on account of malfunctioning of regular spillway gates, damage to the regular spillway, shutdown of outletworks, or the occurrence of two floods in quick succession.

A dam always has some storage capacity above its normal storage level. This storage capacity is termed surcharge storage. If a dam could be made so high as to provide ample
surcharge storage to contain the entire volume of the incoming flood, theoretically no spillway, except an emergency type, would be needed provided that the outlet capacity of the dam can evacuate the surcharge storage well before the arrival of the next flood. Such an ideal situation permitting retention of entire incoming flood by surcharge storage, however, would never exist. In the absence of surcharge storage, spillway must be sufficiently large to pass the peak flood discharge. In such a situation, the peak rate of inflow is more important than the total volume of the incoming flood. On the other hand, if relatively large surcharge storage can be made economically at a dam, a portion of the incoming flood is retained

temporarily in the surcharge storage of the reservoir and the discharging capacity of spillway can be reduced significantly. Economic considerations will usually require that a reservoir be designed to have reasonable surcharge storage as well.

Using the overflow characteristics of an assumed spillway type and known inflow design flood, the maximum spillway discharge and the maximum reservoir water level can be determined by flood routing. For known maximum spillway discharge, the components of the trial spillway can be designed and a complete layout of the spillway prepared. Cost estimates of the trial spillway and dam can now be made.

All relevant factors of topography, hydrology, geology, hydraulics, design requirements, costs, and benefits must be considered for determining the best combination of storage and spillway capacity for the chosen inflow design flood. Some of these important factors are as follows (1):

The characteristics of the inflow flood hydrograph,

The damages which would result if the inflow flood occurred (a) without the dam, (b) with the dam in place, and (c) after the failure of the dam or spillway.

The effects of various dam and spillway combinations on the upstream and down-stream of the dam on account of the resulting backwater and tail-water effects.

Relative costs of various combinations of storage and spillway capacity including the outlet facilities which can be utilised for the duration of the flood.

The cost estimates of various combinations of spillway capacity and dam height for the trial spillways will form the basis for the selection of an economical spillway and the optimum combination of surcharge storage (or the height of the dam) and the spillway capacity.

FLOOD ROUTING

The change in storage Δs of a reservoir depends on the difference between the amount of inflow and outflow and can be expressed by the equation,

$$\Delta s = Q_i \Delta t - Q_o \Delta t \qquad (17.1)$$

where, Δs is the change in storage during time interval Δt, and Q_i and Q_o are, respectively, the average rates of inflow and outflow during the time interval Δt. The rate of inflow at any

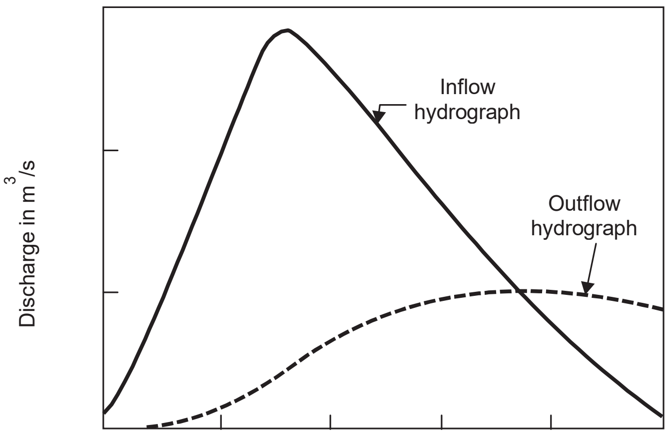
time is obtained from the inflow flood hydrograph selected for the purpose (Fig. 17.1). The rate of outflow (which should, strictly speaking, include outflow from the river outlets, irrigation outlets, and power turbines) is obtained from the outflow discharge versus reservoir water surface elevation curve (Fig. 17.2). Similarly, the storage is obtained by the reservoir storage versus reservoir water surface elevation curve (Fig. 17.3). While the inflow flood hydrogrpah and the storage curve would remain fixed for a given project site, the spillway discharge curve would depend not only on the size and type of the spillway but also on the manner in which the spillway and outlets, in some cases, are operated to regulate the outflow. If one could establish simple mathematical expression for these three curves, a solution of flood routing

3000

2000

1000

0



0 2 4 6 8 10

Time in hours

Fig. 17.1 Typical hydrographs of inflow and outflow floods

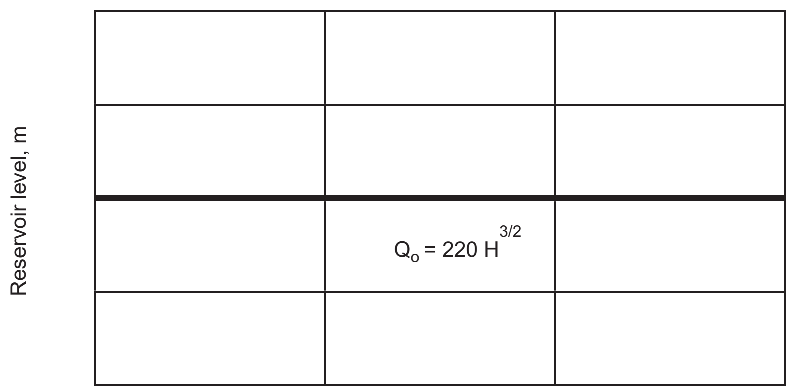
108

106

104

102

100



0 1000 2000 3000

3

Outflow rate, m /s

Fig. 17.2 Spillway discharge versus reservoir level

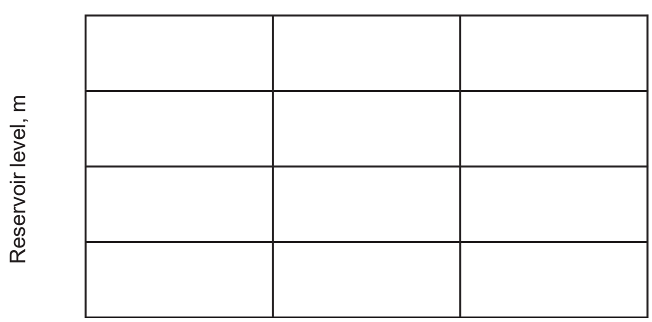
108

106

104

102

100



100 150

200 250

Reservoir storage, 10^6 cu.m.

Fig. 17.3 Reservoir level versus reservoir capacity could be obtained simply by mathematical integration. This, however, is not possible and one has to use one of the several techniques of flood routing ranging from purely arithmetical method to an entirely graphical solution. One such simple arithmetical trial and error method makes the flood routing computations (Table 17.1) in the following manner:

Select a suitable time interval Δt (Col. 2),

Obtain inflow rates for different times from the inflow hydrograph (Fig. 17.1) and enter these values in Col. 3.

Average inflow rates for time interval Δt are entered in Col. 4.

Determine the amount of inflow volume in time and enter the value in Col. 5.

Assume a trial reservoir water surface elevation (Col. 6).

Obtain the rate of outflow from Fig. 17.2 for the assumed trial reservoir water sur- face elevation and enter its value in Col. 7.

Obtain the average rate of outflow for the time interval under consideration and enter this value in Col. 8.

Obtain the amount of outflow volume in time Δt and enter the value in Col. 9.

Obtain the change in storage Δs by subtracting outflow volume from inflow volume (Col. 10).

Add Δs of Col. 10 to the total reservoir storage at the beginning of the time interval (Col. 11).

Determine the reservoir water surface elevation using Fig. 17.3 and enter the same in Col. 12.

Compare the reservoir water surface elevation (Col. 12) with the trial reservoir wa- ter surface elevation (Col. 6). If they do not match within specified accuracy, say 3 cm, make another trial and repeat until the agreement is reached.

Based on the above procedure, flood routing computations have been carried out for the curves of Figs. 17.1-17.3 as shown in Table 17.1. Using the values of the outflow rate (Col. 7), the outflow hydrograph can be prepared as shown in Fig. 17.1. Obviously, the volume indicated by the area between the inflow and outflow hydrographs would be the surcharge storage.

Another method of routing a flood is a graphical method which is also known as inflow-storage-discharge curves method or, simply, ISD method. Equation (17.1) can, alternatively, be written as

$$s_{n+1} - s_n = \frac{Q_{i,n} + Q_{i,n+1}}{2}\Delta t - \frac{Q_{o,n} + Q_{o,n+1}}{2}\Delta t \qquad (17.2)$$

in which, $Q_{i,n}$ and $Q_{o,n}$ represent, respectively, the inflow and outflow discharge rates at the beginning of the n^{th} time step (or, at the end of $(n-1)^{th}$ time step) of duration Δt during which the reservoir storage volume changed by $(s_{n+1} - s_n)$. Equation (17.2) is rewritten as

$$\left(\frac{2s_{n+1}}{\Delta t} + Q_{o,\,n+1}\right) = \left(Q_{i,\,n} + Q_{i,\,n+1}\right) + \left(\frac{2s_n}{\Delta t} - Q_{o,\,n}\right) \qquad (17.3)$$

For this method, one would require a graph $\left(\frac{2s_n}{\Delta t} + Q_o\right)$ versus outflow discharge rate

H ?t K

Q_o , Fig. 17.4. This graph can be prepared by knowing the values of s and Q_o for different values of reservoir water surface elevation (Figs. 17.2 and 17.3) and suitably chosen time interval, ?t. Procedure for computation of the outflow hydrograph is as follows :

Table 17.1 Flood routing computations

Time t (*hrs*)	*Time interval* Δt (*hrs*)	*Inflow rate at time t* (m^3/s)	*Average inflow rate* Q_i *for* Δt (m^3/s)	*Inflow volume during* Δt ($10^4\ m^3$)	*Trial reservoir level at time t* (*m*)	*Outflow rate* Q_0 *at time t* (m^3/s)	*Average outflow rate* Q_0 *for* Δt (m^3/s)	*Outflow volume during* Δt ($10^4\ m^3$)	*Change in storage* Δs *during* Δt ($10^4\ m^3$)	*Total storage at the end of t* ($10^4\ m^3$)	*Reservoir level at the end of t* (*m*)	*Remarks*
1	*2*	*3*	*4*	*5*	*6*	*7*	*8*	*9*	*10*	*11*	*12*	*13*
0		100				0				11000.00		
	1		475	171.0	100.2	19.68	9.84	3.54	167.46	11167.46	100.20	Okay
1		850										
	1		1340	482.4	100.4	55.66	37.67	13.56	468.84	11636.30	100.75	High
2		1830			100.7	128.85	74.27	26.74	455.66	11623.12	100.70	Okay
	1		2310	831.6	101.0	220.00	174.43	62.80	768.80	12391.92	101.42	High
3		2790			101.4	364.43	246.64	88.79	742.81	12365.93	101.40	Okay
	1		2695	970.2	101.80	531.29	447.86	161.23	808.97	13174.90	102.00	High
4		2600			102.00	622.25	493.34	177.60	792.60	13158.53	102.00	Okay
	1		2360	849.6	102.50	869.63	745.94	268.54	581.06	13739.59	102.40	Low
5		2120			102.35	792.55	707.40	254.66	594.94	13753.47	102.35	Okay
	1		1880	676.8	103.00	1143.15	967.85	348.43	328.37	14081.84	102.55	Low
6		1640			102.60	922.32	857.44	308.68	368.12	14121.59	102.60	Okay
	1		1405	505.8	102.70	976.04	949.18	341.71	164.09	14285.68	102.70	Okay
7		1170										
	1		950	342.0	102.70	976.04	976.04	351.37	−9.37	14276.31	102.70	Okay
8		730										
	1		540	194.4	102.60	922.32	949.18	341.71	−147.31	14129.00	102.60	Okay
9		350										
	1		215	77.4	102.50	869.63	895.98	322.55	−245.15	13883.85	102.45	Low
10		80			102.45	843.67	883.00	317.88	−240.48	13888.52	102.45	Okay

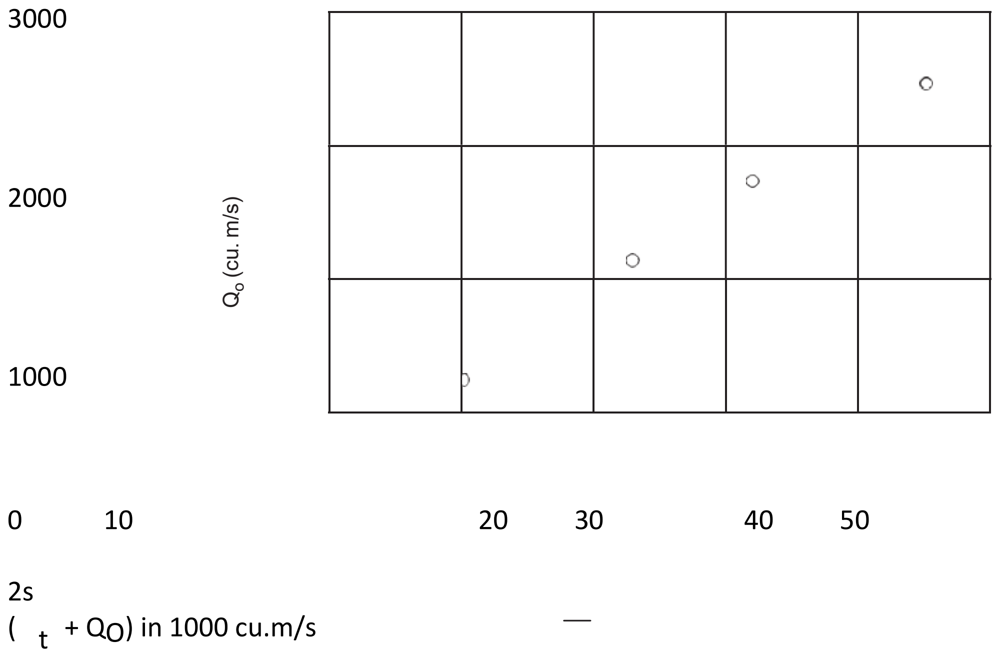


Fig. 17.4

At the beginning of the first time interval (i.e., the routing period), both s_1 and $Q_{o,1}$ are equal to zero if the storage dealt with in this interval is treated as the storage above the spillway crest level. Therefore, Eq. (17.3) yields

$$\frac{2s_2}{\Delta t} + Q_{o,2} = (Q_{i,1} + Q_{i,2})$$

Since both $Q_{i,1}$ and $Q_{i,2}$ are known (Fig. 17.1), $\frac{2s_2}{\Delta t} + Q_{o,2}$ is determined.

Read the value of $Q_{o,2}$ (i.e., the outflow rate at the end of the first time interval, i.e., $t = \Delta t$) for known value of $\frac{2s_2}{\Delta t} + Q_{o,2}$ from Fig. 17.4.

obtain $(\frac{2s_2}{\Delta t} - Q_{o,2})$ (i.e., the value of $(\frac{2s_2}{\Delta t} - Q_o)$ at the beginning of the second time interval) which is equal to $[(\frac{2s_2}{\Delta t} + Q_{o,2}) - 2Q_{o,2}]$.

For the second time interval $\left(\frac{2S_3}{\Delta t} + Q_{o,3}\right) = \left(Q_{i,2} + Q_{i,3}\right) + \left(\frac{2S_2}{\Delta t} - Q_{o,2}\right)$

Since, R.H.S. is known, one can determine L.H.S., the corresponding $Q_{o,3}$ (i.e., the outflow rate at the end of the second time interval i.e., $t = 2\Delta t$), and the value of $\left(\frac{2S_3}{\Delta t} - Q_{o,3}\right)$ i.e., the value of $\frac{2S}{\Delta t} - Q_o$ at the beginning of the third time interval ($t = 2\Delta t$).

Repeat step (iii) for subsequent time steps till the end of the last time interval.
Flood routing problem of Table 17.1 has been solved using ISD method in Tables 17.2 and 17.3 and Figs. 17.4 and 17.5.

Table 17.2 : Computation of $\left(\frac{2s}{\Delta t} + Q_o\right)$ for Q_o with Δt = 3600 sec.

Elevation	Outflow rate (Q_o) m^3/s	Storage m^3	Storage above crest level (s) m^3	$\frac{2s}{\Delta t} + Q_o$
100	0.00	110×10^6	0	0
101	220.00	128×10^6	18×10^6	10220
102	622.25	134×10^6	24×10^6	13956
103	1143.15	149×10^6	39×10^6	22810
104	1760.00	165×10^6	55×10^6	32316
105	2459.68	186×10^6	76×10^6	44682
106	3233.33	215×10^6	215×10^6	61567

Table 17.3 : Computation of Q_o

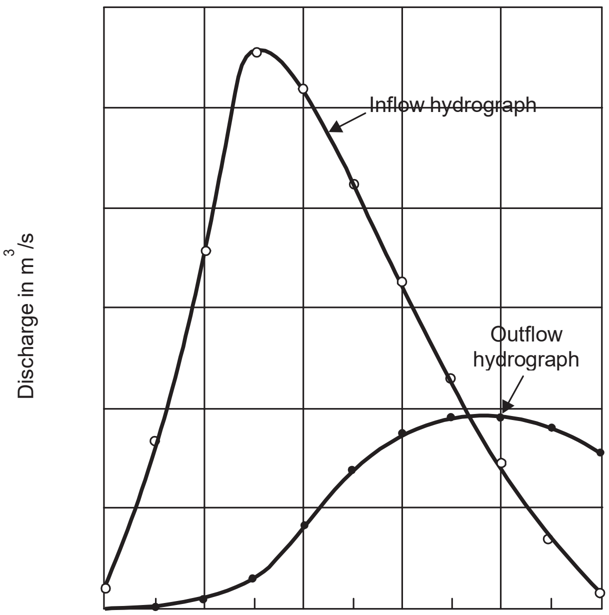
Time (hr)	Inflow rate	$Q_{i,n} + Q_{i,n+1}$	$\left(\frac{2s}{\Delta t} - Q_{o,n}\right)$*	$\frac{2s}{\Delta t} + Q_{o,n+1}$	$Q_{o,n+1}$	Reservoir level
0	100		0	–	–	
1	850	950	930	950	10	100.127
2	1830	2680	3510	3610	50	100.372
3	2790	4620	7830	8130	150	100.775
4	2600	5390	12370	13220	425	101.551
5	2120	4720	15690	17090	700	102.163
6	1640	3760	17700	19450	875	102.510
7	1170	2810	18610	20510	950	102.652
8	730	1900	18610	20510	950	102.652
9	350	1080	17890	19690	900	102.558
10	80	430	16770	18320	775	102.315

* $\frac{2s}{\Delta t} + Q_{o,n+1} - 2Q_{o,n+1}$

3000

2000

1000



0

0 2 4 6 8 10

Time in hours

Fig. 17.5

Example 17.1 A flood enters a reservoir at 0 hours. The ordinates of the inflow hydrograph are as follows:

Hour	0	1	2	3	4	5	6	7
Discharge, Q(m^3/s)	0	100	200	300	400	500	400	300

The area of the waterspread increases linearly from 0.7×10^6 m^2 at the spillway crest level to 1.1×10^6 m^2 at 4 m above the crest level. The effective length of the spillway is 150 m and the coefficient C in the discharge equation $Q = C\,L\,H^{3/2}$ is 2.0 in S.I. units. Using a single time step from 0 to 4 hours and assuming that water is at the crest level at the beginning of the flood, find the reservoir level at 4 hours.

Solution: Let reservoir level be H (m) above the spillway crest at 4 hours from the beginning of the flood. Therefore, Eq. (17.2) reduces to

$$\frac{0+400}{2}\times(4\times3600)-\frac{0+300\,H^{3/2}}{2}\times(4\times3600)$$

$$=\left[0.7\times10^6+\left(0.7\times10^6+\left\{\frac{1.1-0.7}{4}\right\}\times10^6\,H\right)\right]\frac{H}{2}$$

$$5\,H^2+216\,H^{3/2}+70\,H=288$$

$\therefore$ $H = 0.9925$ m

Example 17.2 The pond upstream of a power house may be approximated as a rectangular channel of width 80 m and length 2.0 km. The inflow as well as outflow into the pond at the beginning was 100 m^3/s. The inflow increases gradually to 200 m^3/s in two hours. Assuming that the outflow is through a sluice gate and discharge through which is expressed as Q = 80 $\sqrt{H}$ (in which, H is the head of water above the sill of the sluice gate in the pond), determine the head, H_1 of water above the sill of the sluice gate at the beginning of the increased inflow and at two hours since the beginning.

Solution: Let H be the head of water at the beginning of the increased inflow. Using $Q = 80\sqrt{H}$,

$$100 = 80\sqrt{H_1}$$

$$H_1 = 1.5625 \text{ m}$$

Using Eq. (17.2),

$$\frac{100 + 200}{2} \times (2 \times 3600) - \frac{100 + 80\sqrt{H_2}}{2} \times (2 \times 3600) = 80 \times 2000\,(H_2 - 1.5625)$$

in which, H_2 is the head of water at 2 hours since the beginning of the increased flow. Thus,

$$80 H_2 + 144\sqrt{H_2} = 485$$

$$\therefore \quad H_2 = 2.96 \text{ m}$$

COMPONENTS OF SPILLWAY

The main components of a spillway are: (i) control structure, (ii) conveyance structure, (iii) terminal structure, and (iv) entrance and exit channels.

Control Structure

The control structure of a spillway regulates and controls the outflow from the reservoir. It is usually located at the upstream end of the spillway and consists of some form of orifice or overflow crest. In some cases, however, the control may be at the downstream end. For example, in a 'morning glory' spillway, the downstream tunnel rather than the crest of orifice controls the flow at higher heads. In plan, the outflow crest can be straight, curved, U-shaped, semicircular, or circular. The crest can be sharp, broad, ogee-shaped, or of some other cross- section. Similarly, orifice can have different shapes and may be placed in a horizontal, vertical, or inclined position. Orifices too can be sharp-edged, round-edged or bellmouth-shaped.

Conveyance Structure

The outflow released through the control structure is usually conveyed to the downstream river channel through a discharge channel or waterway. Free fall spillways, however, do notrequire any such conveyance structure. The conveyance structure can be the downstream face of the dam (if the spillway has been constructed in the main body of the dam), or an open channel excavated along the ground surface of one of the abutments, or an underground tunnel excavated through one of the abutments. The conveyance structure too can have a variety ofcross-section depending upon the geologic and topographic characteristics of the site and hydraulic requirements.

Terminal Structure

When water flows from the reservoir level to the downstream river level, the static energy isconverted into kinetic energy which, if not properly dissipated, may cause enough scour, nearthe toe of the dam, that can damage the dam, spillway, and other structures. Therefore, suitable stilling basins at the downstream end of the spillway are usually provided so that the excess kinetic energy is dissipated, and the discharge into the river does not result in objectionable scour. The excess kinetic energy can be dissipated by a hydraulic jump basin or a roller bucket or some other suitable energy dissipator or absorber. In some cases, however, the overflowing water may be delivered directly to the stream if the stream bed consists of erosion-resistant bed rock. The incoming jet should always be projected some distance downstream from the end of the structure by means of such structures as flip buckets or cantilevered extensions.

Entrance and Exit Channel

Entrance channel conveys water from the reservoir to the control structure while the exit channel conveys flow from the terminal structure to the stream channel downstream of the dam. Entrance and exit channels are, however, not required because such spillways draw water directly from the reservoir and discharge it directly into the stream channel as, for example, in case of the overflow spillway in a concrete dam. In spillways which are placed along the abutments or located near saddles or ridges, entrance and exit channels would be needed.

TYPES OF SPILLWAY

Spillway is usually referred to as controlled or uncontrolled depending on whether spillway gates for controlling the flow have been provided or not. A free or uncontrolled spillway automatically releases water whenever the reservoir level rises above the overflow crest level. The main advantage of an uncontrolled spillway is that it does not require constant attendance and operation of the regulating devices by an operator. Besides, there are no problems related to the maintenance and repair of the devices. If it is not possible to provide a sufficiently long uncontrolled spillway crest or obtain a large enough surcharge head to meet the requirements of spillway capacity, one has to provide regulating gates. Such gates enable release of water, if required, even when the reservoir level is below the normal reservoir water surface level. Most common types of spillway are as follows:

Free overfall (straight drop) spillway,
Ogee (overflow) spillway,
Side-channel spillway,
Chute (or open channel or trough) spillway,
Shaft (or morning glory) spillway,
Siphon spillway,
Cascade spillway, and
Tunnel (conduit) spillway.

Free Overfall Spillway

As the name indicates, the flowing water drops freely from the crest of a free overfall spillway. At times, the crest is extended in the form of an overhanging lip to direct small discharges away from the downstream face of the overflow section. The underside of the falling water jet is properly ventilated so that the jet does not pulsate. Such a spillway is better suited for a thin arch dam whose downstream face is nearly vertical. Since the flowing water usually drops into the stream bed, objectionable scour may occur in some cases and a deep plunge pool may be formed. If erosion cannot be tolerated, plunge pool is created by constructing an auxiliary dam downstream of the main dam. Alternatively, a basin is excavated and is provided with a concrete apron. When tail-water depth is sufficient, a hydraulic jump forms when the water jet falls upon a flat apron. Free overfall spillways are restricted only to situations where the hydraulic drop from the reservoir level to tail-water level is less than about 6 m.

Ogee (Overflow) Spillway

An ogee spillway has a control weir whose profile is as shown in Fig. 17.6. The upper part of the spillway surface matches closely with the profile of the lower nappe of a ventilated sheet of water falling freely from a sharp-crested weir. The lower part of the spillway surface is tangential to the upper curve and supports the falling sheet of water. The downstream end of the spillway is in the form of a reverse curve which turns the flow into the apron of a stilling basin or into the spillway discharge channel. An ogee spillway is generally used for concrete and masonry dams. It is ideally suited to wider valleys where sufficient crest length may be provided.

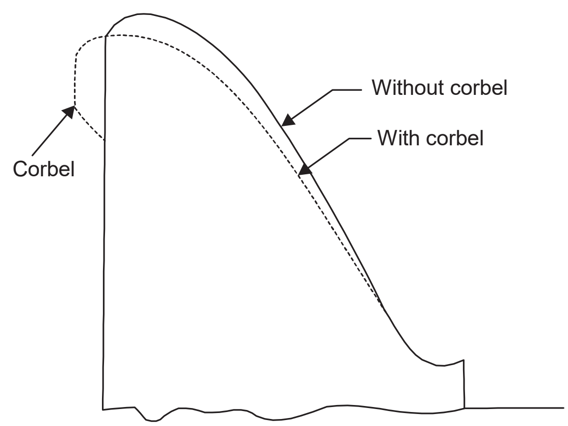


Fig. 17.6 Ogee spillway with an overhang

The profile of an ogee spillway is designed for a given design discharge (or corresponding design surcharge head or, simply, design head). When the flowing discharge equals the design discharge, the flow adheres to the spillway surface with minimum interference from the boundary surface and air has no access to the underside of the water sheet. The discharge efficiency is maximum under such condition and the pressure along the spillway surface is atmospheric. If the flowing discharge exceeds the design discharge, the water sheet tends to pull away from the spillway surface and thus produces sub-atmospheric pressure along the surface of the spillway. While negative pressure may cause cavitation and other problems, it increases the effective head and increases the discharge. On the other hand, positive hydrostatic pressure will occur on the spillway surface, if the flowing discharge is less than the design discharge.

Model tests have indicated that the design head may be safely exceeded by about 50% beyond which cavitation may develop. Therefore, spillway profle may be designed for 75% of the peak head for the maximum design flood.

An upstream overhang, known as corbel, is added to the upstream face of the spillway as shown in Fig. 17.6. The effect of the corbel is to shift the nappe (and, hence, the spillway profile) backward which results in saving of concrete. If the height of the vertical face of the corbel is kept more than 0.3 times the head over the crest, the discharge coefficient of the spillway will be practically the same as it would be if the vertical face of the corbel were to extend to the full height of the spillway.

The shape of an ogee crest, approximating the profile of the underside of a water jet flowing over a sharp-crested weir, depends on: (i) the head, (ii) the inclination of the upstream face of the overflow section, and (iii) the height of the overflow section above the floor of the entrance channel which affects the velocity of approach to the crest. A simple shape of an ogee crest suitable for dams with vertical upstream face is shown in Fig. 17.7. It consists of an upstream surface shaped as an arc of a circle upto the apex of the crest followed by a parabolic downstream surface. This type of ogee crest is suitable for preliminary estimate and for final designs when a more refined shape is not required (2).

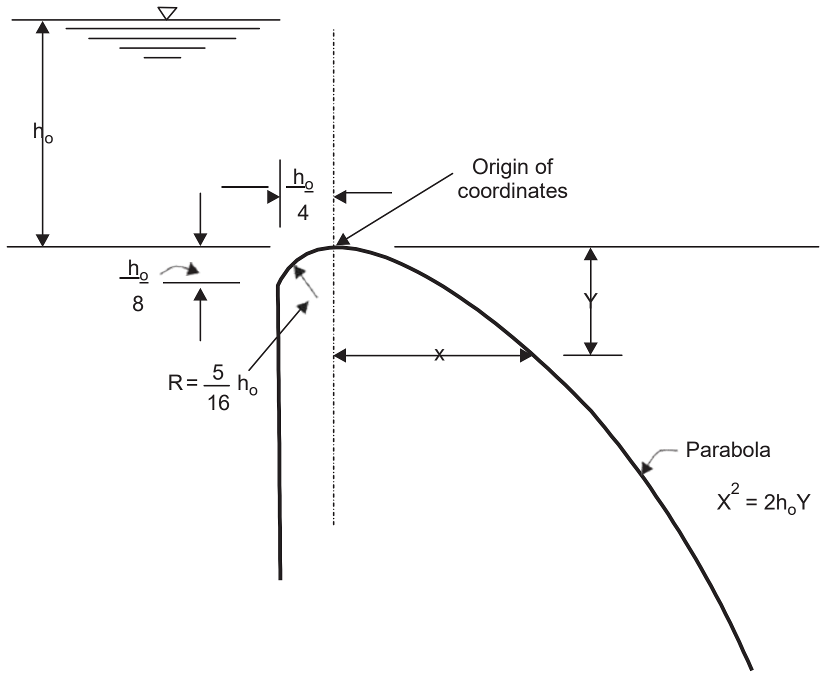


Fig. 17.7 Ogee crest for spillway with vertical upstream face

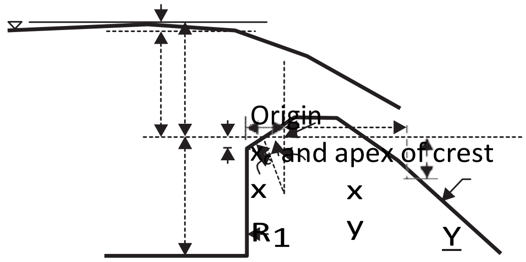
An extensive study of crest shapes has been made by USBR (3). These crest shapes can be represented by the profile shown in Fig. 17.8 (a) and defined with respect to the coordinate axes at the apex of the crest. The part of the profile upstream of the apex of the crest consists of either a single curve and a tangent or a compound circular curve. The profile downstream of the apex of the crest is defined by the equation

$$\frac{y}{H_0} = -k\left(\frac{x}{H_0}\right)^n \qquad (17.4)$$

in which, H_0 is the total head on the crest including the velocity of approach and the constant

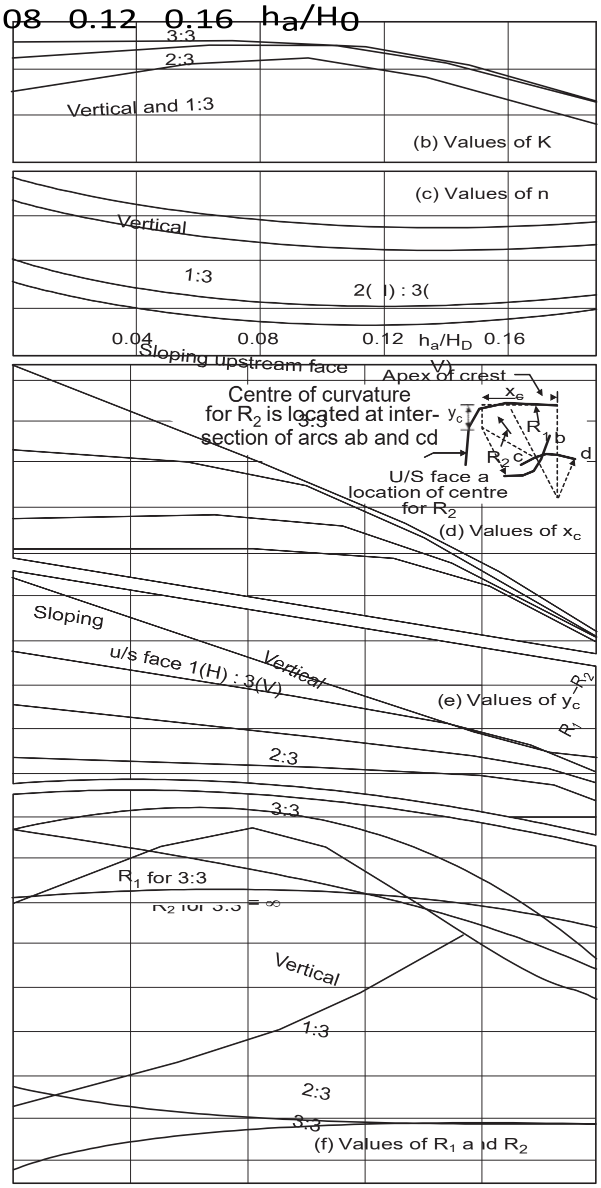
k and n depend on: (i) the inclination of the upstream face, and (ii) the velocity of approach as

has been shown in Fig. 17.8 (b) and 17.8 (c). Similarly, the value of X_C, Y_C, R_1, and R_2, shown as elements of the crest profile in Fig. 17.8 (a), can be obtained from Fig. 17.8(d), 17.8 (e), and 17.8 (f).



Upstream face

Elements of ogee - shaped crest profiles



0.45

0.40

0.35

0.30

0.25

0.20

0.15

0 0.04 0.08 0.12 0.16 0.20

h_a/H_0

R/H_0

R_1 for 2:3

with R

R_2 for 2:3

R_2 for vertical

R_2 for 1:3

Fig. 17.8 Parameters for ogee-shaped crest profile

The discharge Q flowing over an ogee crest of effective length L can be obtained from the formula

$Q = CLH_o^{3/2}$ (17.5)

in which, C is a variable coefficient of discharge which depends on: (i) depth of approach [Fig.
17.9 (a)], (ii) slope of the upstream face [Fig. 17.9 (b)], (iii) position of the downstream apron [Fig. 17.9 (c)], (iv) the downstream tail-water level, [Fig. 17.9 (d)], and (v) the relation between the actual crest shape (corresponding to the design head) and the ideal nappe shape (corresponding to the actual head of flow) [Fig. 17.9 (e)]. The effect of the downstream apron and tail-water level would generally be felt when ogee crest is being used as control structure for a side-channel spillway and the tail-water level is high enough to affect the discharge, i.e., the crest is submerged. For usual ogee spillways, this situation would not arise.

The effective length of the crest L will be less than the total length of the crest on account of side contractions whenever crest piers are provided. The effective length L and total length L⍰ of a crest are related as follows (1):

L = L⍰ – 2 (N⍰K_p + K_a) H_o (17.6)

Here, N⍰ is the number of piers, and K_p and K_a are, respectively, pier and abutment coefficients. Values of K_p vary between zero (for pointed nose piers) and 0.02 (for square-nosed pier with suitable rounded corners). Values of K_a vary between zero and 0.2.

2.2

2.1

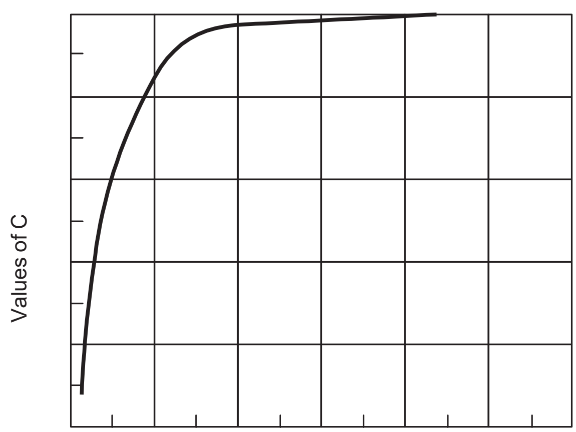
2.0

1.9

1.8

1.7

0



2 4 6 8 10 12

P/H_O

(a) Variation of coefficient of discharge for ogee-shaped crest with vertical uptream face

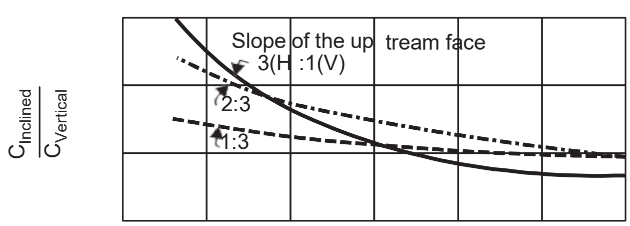
1.04

1.02

1.00

0.98

0

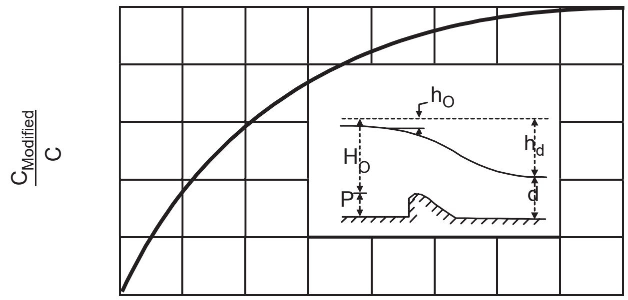


0.5 P/H_o 1.0 1.5

(b)- Coefficient of discharge for ogee- shaped crest with sloping uptream face

Fig. 17.9 (a and b). Variations of coefficient of dischrge for ogee spillway (Contd...)

1.00

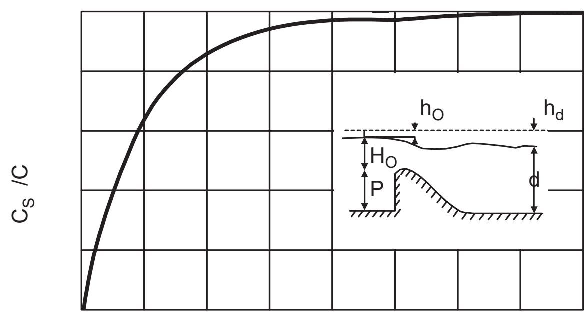


0.90

0.80

0.76
1.0 1.1 1.2 1.3 1.4 1.5 1.6 1.7 1.8
$(h_O + d)/H_O$
(c) Ratio of discharge coefficient due to apron effect

1.0



0.8

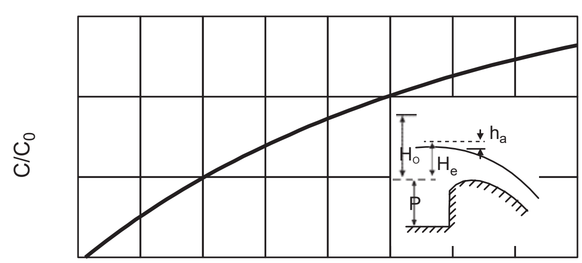
0.6

0.4

0.2

0
0 0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8
h_d
$/H_O$
(d) Ratio of discharge coefficient due to tailwater effect

1.1



1.0

0.9

0.8
0 0.2 0.4 0.6 0.8 1.0 1.2 1.4 1.6
H_e /
H_O
(e) Coefficient of discharge for H_O other than the design head

Fig. 17.9 (c, d and e) Variations of coefficient of discharge for ogee spillway (4)

Example 17.3 Obtain USBR profile for an ogee spillway with the following data: Design discharge = 13875 m^3/s
Crest length of spillway = 183 m Crest level of spillway = 203.34 m River bed level = 166.50 m

Solution: Assuming C = 2.20 in Eq. (17.5), one obtains

$$H_o = \left(\frac{13875}{183 \times 2.20}\right)^{2/3} = 10.59 \text{ m}$$

and $P = 203.34 - 166.50 = 36.84$ m

$$\therefore \quad \frac{P}{H_o} = 3.48$$

From Fig. 17.9 (a), C = 2.18

With C = 2.18, Eq. (17.5) yields $H_0 = 10.655$ m

Design head for spillway profile = 0.75 × 10.655 = 7.99 m

$\approx$ 8.00 m (say)

$$\therefore \quad \frac{P}{H_o} = \frac{36.84}{8.00} = 4.605$$

$$\text{Approach velocity, } v_a = \frac{13875}{183\,(36.84 + 8.00)} = 1.69 \text{ m/s}$$

$\therefore$ $h_a = 0.15$ m and $h_a/H_o = 0.019$ From Figs. 17.8 (b) and (c), k = 0.504 and n = 1.861

Hence, the profile of the ogee spillway, [Fig. 17.8 (a)] is given by

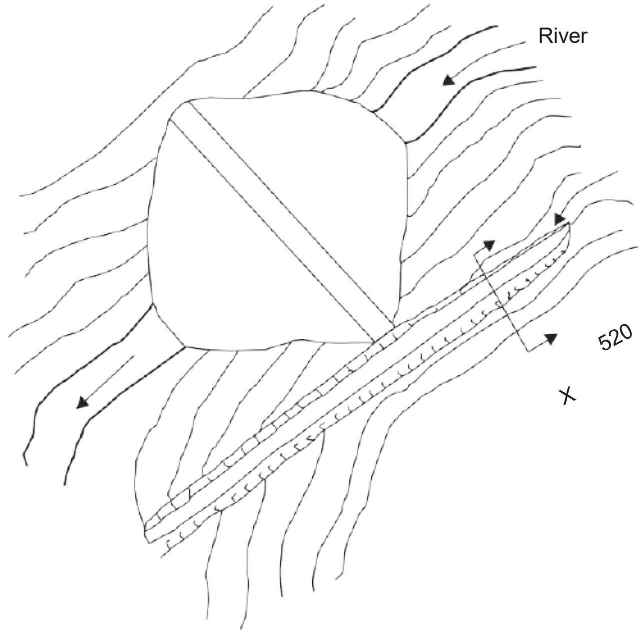
$$\frac{y}{8.00} = -0.504\left(\frac{x}{8.00}\right)^{1.861}$$

SIDE-CHANNEL SPILLWAY

The control weir of a side-channel spillway (Fig. 17.10) is located alongside and approximately parallel to the upstream portion of the spillway discharge channel which itself may be either an open channel, a closed conduit, or an inclined tunnel. The spillway discharge flows over the weir crest and falls into a narrow trough (i.e., upstream of the discharge channel) and takes an approximately 90°-turn before continuing into the spillway discharge channel. The control structure, in plan, may be straight, curved, semi-circular or U-shaped. The overflow section may be broad-crested instead of ogee-shaped.

One should note that when the flow in the side channel trough is subcritical, the incomingflow from the control structure (i.e., the overflow crest) will not cause high transverse velocities because of the low drop due to relatively higher depth of flow in the trough. This would effect good diffusion and intermingling of the incoming flow with the trough water due to relatively low velocities of both the incoming flow and the flow in the trough. Therefore, there would be comparatively smooth flow in the side channel trough. However, when the flow in the side channel trough is supercritical, the flow velocities in the trough would be high and the depth of flow small, causing the incoming flow to have a relatively higher drop. Therefore, the intermixing of the high-energy transverse flow with the trough stream will be rough and turbulent producing violent wave action causing vibrations. Therefore, the flow in the side channel trough should be maintained at subcritical condition for good hydraulic performance.

Moreover, the amount of excavation would also increase for larger bed widths. While the flow in the side channel trough should preferably be subcritical, the flow in the discharge channel is supercritical and a control section downstream of the trough is provided by eitherconstricting the channel width or raising the channel bottom.

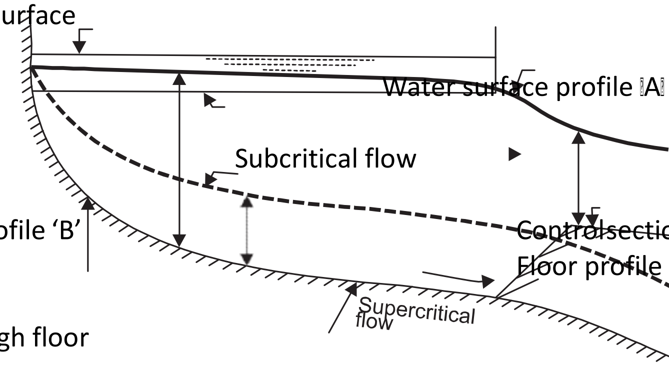


Plan

Reservoir water surface

Overflow crest
Subcritical
depth _______
Water surface profile 'B'
Subcriticaldepth

Side channeltrough floor
profile Discharge channel
floor profile 'B'

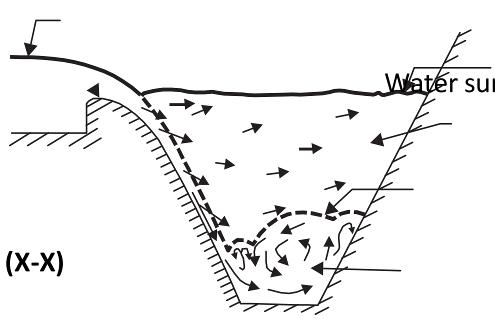


Side channel profile

Reservoir water surface

Overflow______ crest
Water surface 'B'

Supercritical flow



Side channel cross-section (X-X)

Fig. 17.10 Side-channel spillway

Because of spatial flow conditions, the depth of flow in the side-channel trough would never be the same at different sections. For any short reach of channel Δx, the change in water surface Δy can be determined by either of the following equations (4):

$$\Delta y = \frac{Q_1(v_1 + v_2)}{g(Q_1 + Q_2)}\left[(v_2 - v_1) + \frac{v_2(Q_2 - Q_1)}{Q_1}\right] \qquad (17.7)$$

$$\Delta y = \frac{Q_2(v_1 + v_2)}{g(Q_1 + Q_2)}\left[(v_2 - v_1) + \frac{v_1(Q_2 - Q_1)}{Q_2}\right] \qquad (17.8)$$

Here, Q_1 and v_1 are the values of discharge and velocity, respectively, at the beginning of the reach, and Q_2 and v_2 are those values at the end of the reach. For free flow conditions, the behaviour of a side-channel spillway is similar to that of an overflow spillway and is dependent on the profile of the weir crest. For larger discharges, however, the flow over the crest may be submerged and the flow conditions will then be governed by the conditions in the side-channel trough. A side-channel spillway is an ideal choice: (i) for earth or rockfill dams in narrow canyons and for situations where direct overflow is not permissible, (ii) where the space required for a chute spillway of adequate crest length is not available, or (iii) when a long overflow crest is required in order to limit the surcharge head for the design inflow flood. Because of the turbulences and vibrations inherent in side channel flow, a side channel spillway is generally not considered except when a strong foundation (such as rock foundation) exists.

Design of side channel trough involves computation of water surface profile starting from the control section (rectangular in shape and at which the critical depth, velocity and the velocity head are known for given discharge) to the upstream end of the side channel trough. The trough is usually trapezoidal with side slopes of 0.5H:1V and relatively flat bed slope that would provide larger depth and smaller velocities to insure better intermixing of flows in the initial reach of the trough. A cross-section with minimum width – depth ratio will result in the best hydraulic performance (5). However, minimum bed width (say, about 3 m) is required to avoid construction difficulties due to confined working space. There would be a transition between the downstream end of the trough and the control section. The head loss in transition (to include losses due to contraction and friction, and also losses due to diffusion of flows in the trough) is assumed to be equal to 0.2 times the difference in velocity heads between the ends of the transition. The flow characteristics (depth and velocity or velocity head) at the downstream end of the trough (i.e., upstream end of the transition) are obtained by solving Bernoulli's equation. The equation will have to be solved by trial and error. For this, assume a suitable value of the depth of flow at the upstream end of the transition and the corresponding velocity head. If these values satisfy the Bernoulli's equation (applied for the two end sections of the transition), one has obtained the flow characteristics at the upstream end of the transition. Otherwise, one has to assume another trial value and repeat the computations till the Bernoulli's equation is satisfied. Thereafter, the water surface profile along the side channel trough can be determined using either Eq. (17.7) or Eq. (17.8). The channel profile and the water surface profiles are, then, plotted relative to the crest (of the control structure) and the reservoir water level. The maximum submergence at the upstream end of the trough that can be tolerated is about two-third the head over the control structure. If the maximum water surface level in the side channel trough results in submergence more than the permissible value, the end of the side channel trough will have to be lowered.

The control structure of a side channel spillway generally consists of an ogee crest which is designed by the method described in section 17.4.2. Flow in the discharge channel downstream from the control will be the same as that in an ordinary channel or chute spillway.

Chute Spillway

In a chute (or trough) spillway, the spillway discharge flows in an open channel (named as 'chute' or 'trough') right from the reservoir to the downstream river. The open channel can be located either along the abutment of the dam or through a saddle, (Fig. 17.11). The channel bed should always be kept in excavation and its side slope must be designed to be stable with sufficient margin of safety. As far as possible, bends in the channel should be avoided. If it becomes necessary to provide a bend, it should be gentle. The spillway control structure can be an overflow crest, or a gated orifice or some other suitable control device. The control device is usually placed normal or nearly normal to the axis of the chute. The simplest form of a chute spillway is an open channel with a straight centre line and constant width. However, often the axis of either the entrance channel or the discharge channel is curved to suit the topography of the site. The flow condition varies from subcritical upstream of the controlling crest to critical at the crest and supercritical in the discharge channel. The chute spillway is ideally suited with earthfill dams because of: (i) simplicity of their design and construction, (ii) their adaptability to all types of foundation ranging from solid rock to soft clay, and (iii) overall economy usually obtained by the use of large amounts of spillway excavation for the construction of embankment. The chute spillway is also suitable for concrete dams constructed in narrow valleys across a river whose bed is erodible for which the ogee spillway becomes unsuitable.

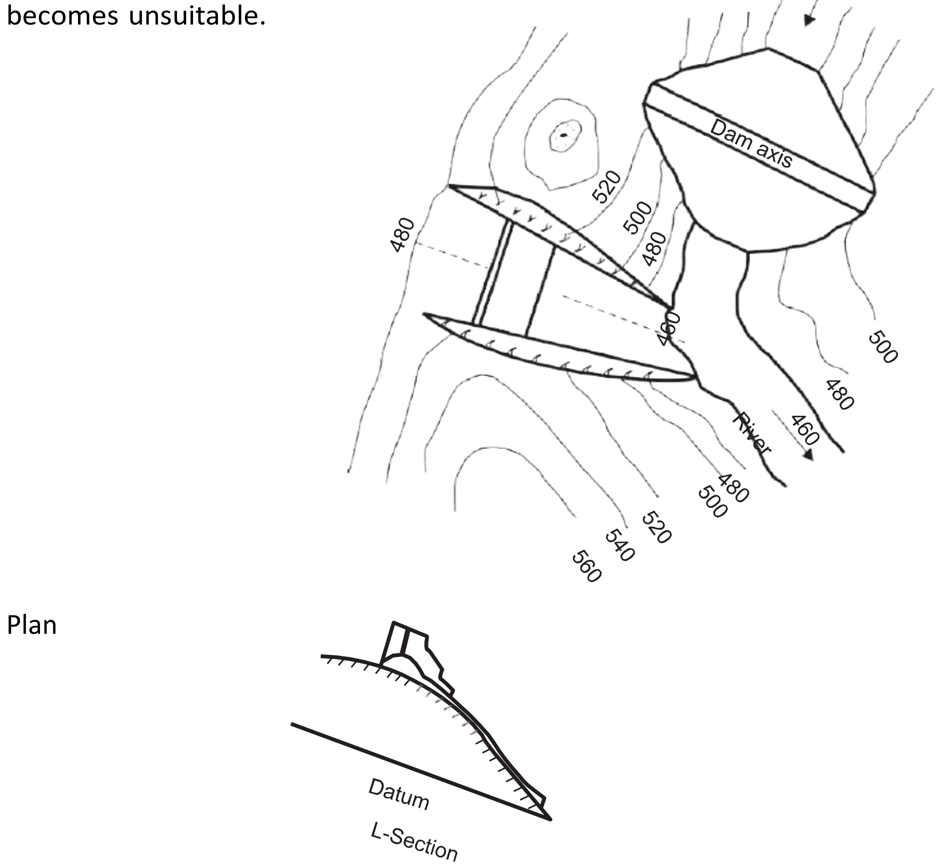


Fig. 17.11 Chute spillway

Shaft Spillway

In a shaft spillway (Fig. 17.12) water enters a horizontal crest, drops through a vertical or sloping shaft, and then flows to the downstream river channel through a horizontal or nearly horizontal conduit or tunnel. A rock outcrop projecting into the reservoir slightly upstream of the dam would be an ideal site for shaft spillway. Depending upon the level of the rock outcrop and the required crest level, a spillway may have to be either constructed or excavated. The diversion tunnels, if used for river diversion purposes during construction, can be utilised for discharge tunnels of the spillway. Radial piers provided on the spillway crest ensure radial flow towards the spillway and also provide support to the bridge which would connect the spillway with the dam or the surrounding hill.

A shaft spillway with a funnel-shaped inlet is called a “morning glory” or “glory hole” spillway. One of its distinguishing characteristics is that near maximum capacity of the spillway is attained at relatively low heads. Therefore, a shaft spillway is ideal when maximum spillway discharge is not likely to be exceeded. Because of this feature, however, the spillway becomes unsuitable when a flow larger than the selected design flow occurs. This disadvantage can be got rid of by providing an auxiliary or emergency spillway and using the shaft spillway as service spillway.

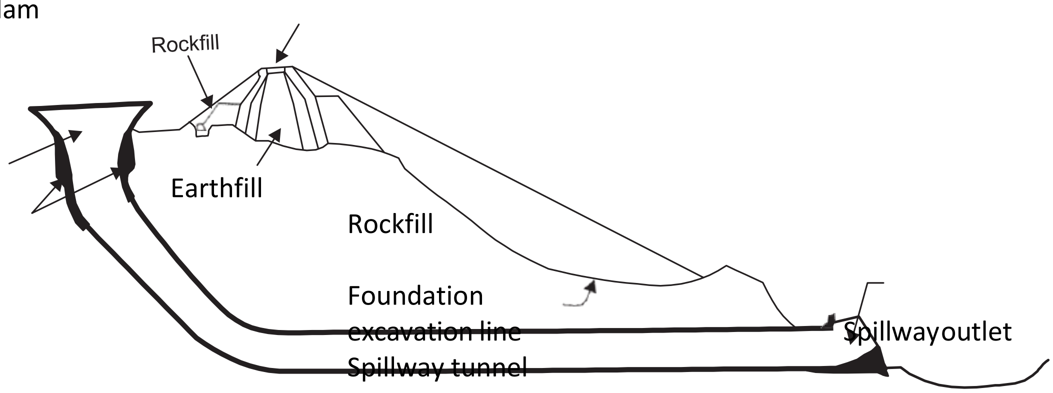


Fig. 17.12 Shaft spillway

Depending upon the type of crest, the shaft spillway can be either standard-crested or flat-crested (Fig. 17.13). In a standard-crested spillway, the water begins its free fall immediately upon leaving the crest whereas in the flat-crested spillway water approaches the crest on a flat slope before beginning its free fall. The standard-crested spillway would have a smaller diameter crest since its coefficient of discharge is greater than that of a flat crest. Therefore, if the shaft spillway is to be constructed in the form of a tower, it would be economical to have a standard- crested spillway. However, a flat-crest shaft spillway has a smaller funnel diameter and is, therefore, more advantageous when the spillway is to be excavated in rock. The design of a standard-crested shaft spillway has been discussed here.

The design of a standard-crested shaft spillway involves the determination of the funnel radius, R, and the head over the theoretical sharp crest, H, for known discharge, Q and the allowable maximum head, h on the spillway crest (Fig. 17.14). The method given by Creager, et al. (6) is valid for negligible velocity of approach and involves trial at two stages and is based on the following equations:

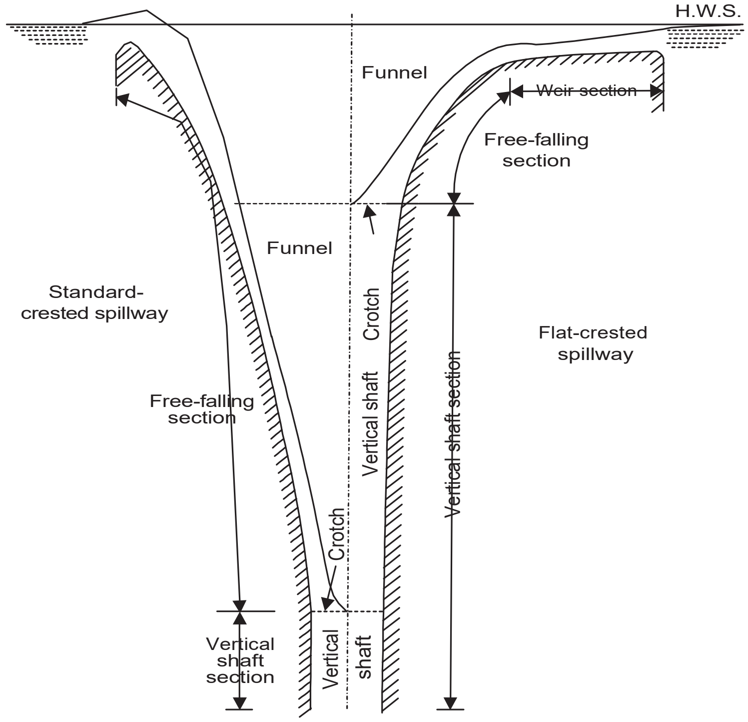
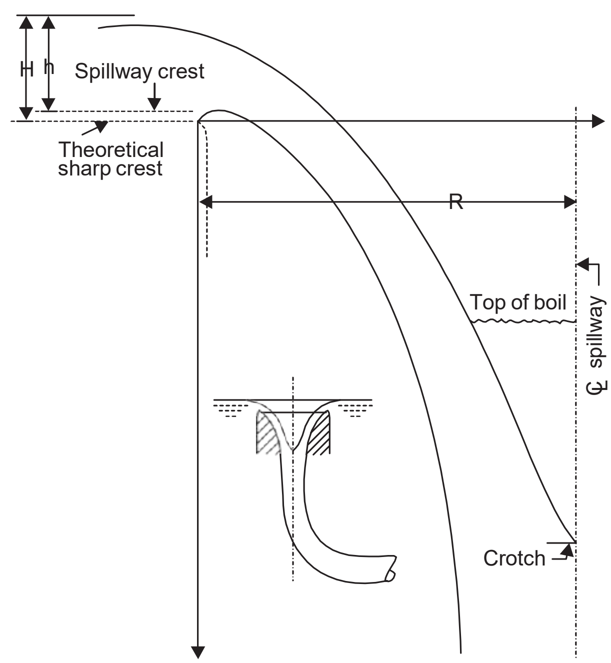


Fig. 17.13 Standard-crested and flat-crested profiles for shaft spillway

x



y

Fig. 17.14 Jet profile over standard-crested shaft spillway

$Q = 2\pi R C_1 \sqrt{g} H^{3/2}$ (17.9)

$\frac{r}{H} = 0.11 - 0.10 \frac{H}{R}$ (17.10)

$H = h + r$ (17.11)

In these equations, C_1 is the coefficient of discharge which is related to H/R as shown in Fig. 17.15, and r is the rise of the lower nappe above the theoretical sharp crest. The different steps involved in the trial method are as follows:

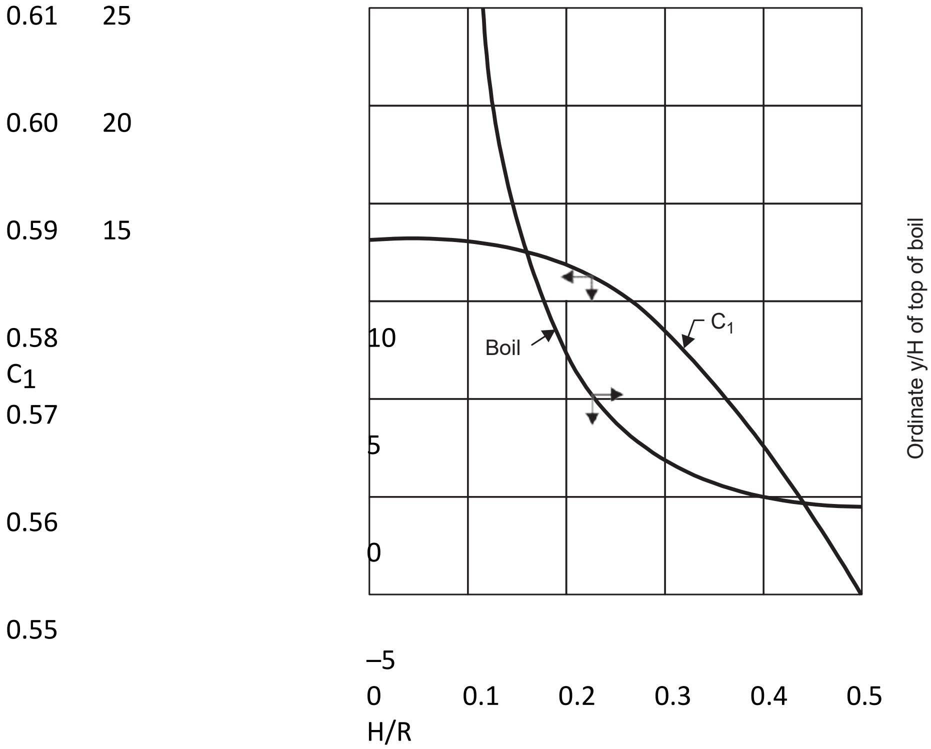


Fig. 17.15 Variations of C_1 and the ordinate y/H of the top of boil for standard-crested shaft spillway (6)

Assume some suitable values of H and R and, hence, H/R, and obtain the value of C_1 from Fig. 17.15.

Determine the discharge intensity, q per unit length of the crest, i.e.,

$q = C_1 \sqrt{g} H^{3/2}$ (17.12)

Obtain the required radius R from

$R = \frac{Q}{2\pi q}$ (17.13)

Compare this value of R with the assumed value of R in step (i). If these two values do not match, assume another value of R for the same assumed value of H and re- peat the procedure until the value of R obtained in step (iii) matches with the as- sumed value of R.

Determine r from Eq. (17.10) and then obtain h from Eq. (17.11). If this value of h does not agree with the given value of h, one has to assume another value of H, andrepeat steps (i) to (iv) until the agreement is reached.

Vittal (7) has obtained a direct solution for R and H by rewriting Eqs. (17.9) to (17.11) as follows:

From Eq. (17.9),

$$Q_* = \frac{Q}{g^{1/2} h^{5/2}} = 2\sqrt{2}\,\frac{R}{h}\,C_1\left(\frac{H}{h}\right)^{3/2} \qquad (17.14)$$

Using Eq. (17.11), Eq. (17.10) can be rewritten in the following two forms:

$$\frac{r}{H} = \frac{H - h}{H} = \left(1 - \frac{h}{H}\right) = 0.11 + 0.10\,\frac{H}{R}$$

$$\frac{h}{H} = 0.89 - 0.10\,\frac{H}{R} \qquad (17.15)$$

and

$$\frac{r}{R} = \frac{H - h}{R} = \frac{H}{R} - \frac{h}{R} = \frac{H}{R}\left(0.11 + 0.10\,\frac{H}{R}\right)$$

$$\left(\frac{H}{R}\right)^2 - 8.9\,\frac{H}{R} + 10\,\frac{h}{R} = 0 \qquad (17.16)$$

From Fig. 17.15, and Eqs. (17.15) and (17.16), one obtains the following functional relationships:

$$C_1 = f_1\left(\frac{H}{R}\right)$$

$$\frac{h}{H} = f_2\left(\frac{H}{R}\right)$$

$$\frac{H}{R} = f_3\left(\frac{h}{R}\right)$$

Using the above functional relationships, Vittal (7) obtained the following functional relation:

$$C_1\left(\frac{H}{h}\right)^{3/2} = f_4\left(\frac{h}{R}\right) \qquad (17.17)$$

Actual relationship of Eq. (17.17) can be obtained by obtaining C_1 from Fig. 17.15, the values of $\frac{h}{H}$ from Eq. (17.15), and $\frac{H}{R}$ from Eq. (17.16) for different values of $\frac{h}{R}$ ranging from 0 to 0.5. One can, therefore, prepare a curve of $C_1\left(\frac{H}{h}\right)^{3/2}$ versus $\left(\frac{h}{R}\right)$. Vittal (7) obtained the following equation for this curve:

$$C_1\left(\frac{H}{h}\right)^{3/2} = 0.6988 + 0.0882\left(\frac{h}{R}\right) + 0.296\left(\frac{h}{R}\right)^2 \qquad (17.18)$$

On combining Eq. (17.18) with Eq. (17.14), and solving the resulting quadratic equation, one obtains,

$$\left(\frac{R}{h}\right)^2 - (0.2280\, Q_* - 0.1263)\frac{R}{h} - 0.3861 = 0$$

$$\therefore \quad \frac{R}{h} = (0.1140\, Q_* - 0.0632)\left[1 + \sqrt{1 + \frac{1.4432}{(0.2280\, Q_* + 0.1263)^2}}\right] \qquad (17.19)$$

For large values of Q_* (say, greater than 25), Eq. (17.19) can be approximated to

$$\frac{R}{h} = 0.2280\, Q_* + 0.1264 \qquad (17.20)$$

For known Q and h and, hence, Q_*, one can easily determine $\frac{R}{h}$ (and, hence, R) from one of the Eqs. (17.19) and (17.20). Using Eq. (17.16) one can determine $\frac{H}{R}$ and, hence, H. For example, values of Q and h equal to 851.2 m^3/s and 3.05 m, respectively, yield

$Q_* = 16.73$

$\frac{R}{h} = 4.03 \quad \therefore \quad R = 12.29$ m

$\frac{H}{R} = 0.27 \quad \therefore \quad H = 3.325$ m

The profile of the underside of the nappe over the circular sharp-crested weir can be determined from Fig. 17.16 which enables computation of x for a given value of y, and already computed R and H. If R_0 is the radius of the lower nappe at any given elevation y, then

$R_0 = R - x$

And x_0, representing the value of x for the upper side of the nappe at a given value of y, is obtained from (6)

$$x_0 = R - \sqrt{R^2 - \frac{Q}{\pi\sqrt{2g(y + 1.269\, H)}}} \qquad (17.21)$$

Here, $y + 1.269\, H$ is the head available for vertical velocity. Proceeding in this manner, one can compute the value of x and x_0 for different values of y until x_0 equals R at which value of y the horizontal velocity ceases and its energy gets converted into a 'boil' as shown in Fig.

The ordinate $\frac{y}{H}$ of the top of the boil can be computed from the curve shown in Fig.

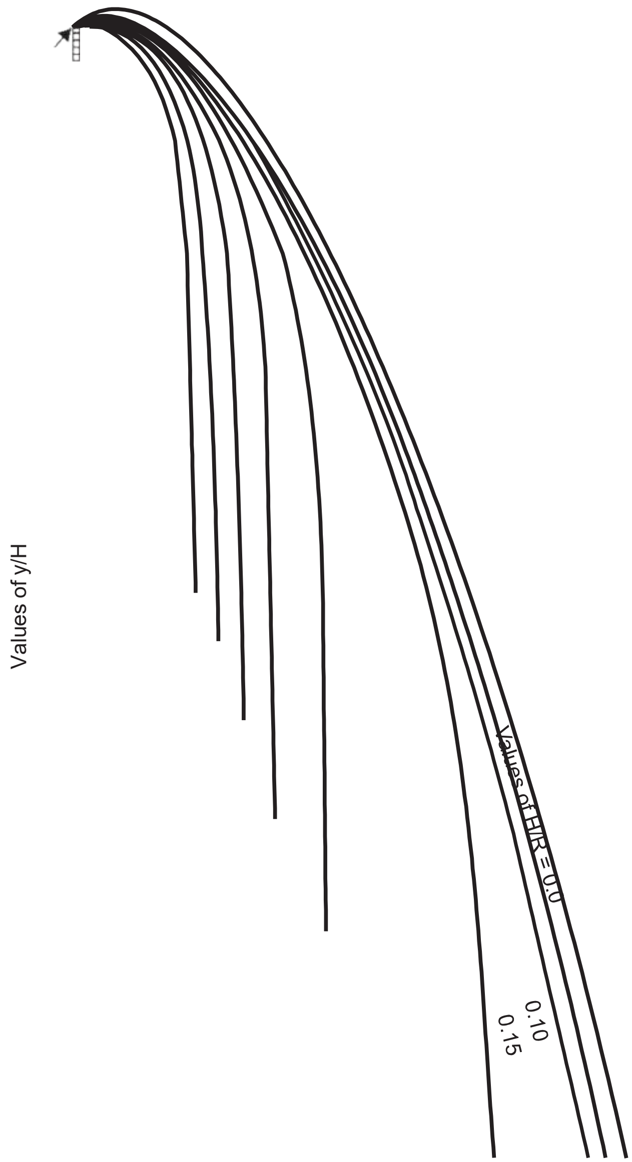
The point at which x_0 becomes equal to R is usually known as 'crotch'. The above analysis does not include the friction loss as it is difficult to be considered and is within permissible limits for the accuracy desired (6).

The diameter of the vertical shaft below the crotch continues to decrease until the size becomes such that the discharge, Q can be carried according to the head available. The radius of the transition shaft, R' at a given elevation y is obtained from

$$Q = \pi R'^2 \sqrt{2 g h_v} \qquad (17.22)$$

in which, $h_v = y + 1.269\, H - h_{L1} - h_{L2}$

where, h_{L1} and h_{L2} are the head losses due to friction, respectively, from the crest to the crotch and from the crotch to the elevation under consideration.



0.0
Crestof
theoreticalweir
1

2

3

4

5

6

7

8

9

10
0 1 2 3 4 5
Values of x/H

Fig. 17.16 Profile of the undernappe of the standard-crested shaft spillway (6)
Siphon Spillway
A siphon spillway (Fig. 17.17) is essentially a closed conduit system which uses the principle of siphonic action. The conduit system is the shape of an inverted U of unequal legs with its inlet

end at or below normal reservoir storage level. When the reservoir water level rises above the normal level, the initial flow of water is similar to the flow over a weir. When the air in the bend has been exhausted, siphonic action starts and continuous flow is maintained until air enters the bend. The inlet end of the conduit is placed well below the normal reservoir water level to prevent ice and drift from entering the conduit. Therefore, once the siphonic action starts, the spillway continues to discharge even after the reservoir water level falls below the normal level. As such, a siphon-breaking air vent is always provided so that siphonic action can be broken once the reservoir water level has been drawn down to the normal level in the reservoir. Siphon spillways can be either constructed of concrete or formed of steel pipe. The thickness of the wall of the siphon structure should, however, be sufficiently strong to withstand the negative pressures which develop in the siphon. Pressure at the throat section (i.e., section 2) can be determined by the use of Bernoulli's equation. Thus,

$$\frac{p_2}{\rho g} + \frac{v_2^2}{2g} + H_1 = \frac{v_3^2}{2g} + h_L$$

where, h_L is the head loss between sections 2 and 3. Therefore,

$$\frac{p_2}{\rho g} = -H_1 + \left(\frac{v_3^2}{2g} - \frac{v_2^2}{2g}\right) + h_L \qquad (17.23)$$

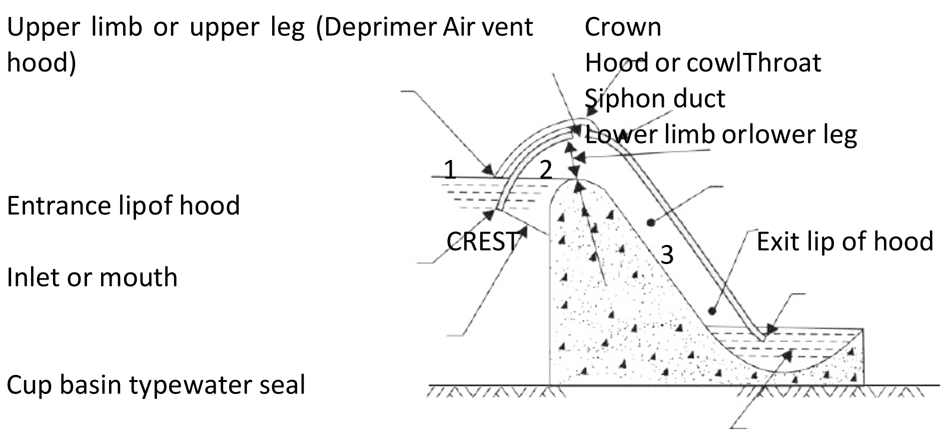
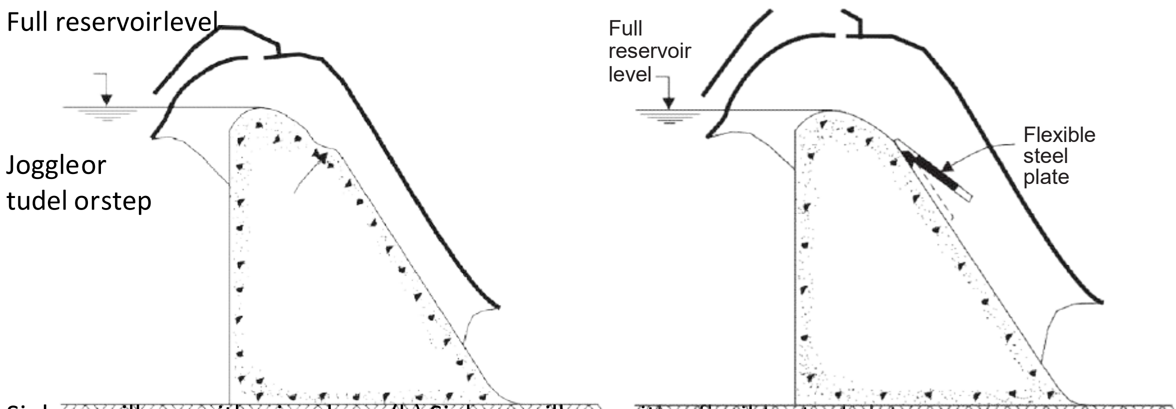


Fig. 17.17 Siphon spillway

If the cross-sectional areas at sections 2 and 3 are the same, $v_3 = v_2$ and $h_L < H_1$, the pressure at the throat is always negative. Besides, the pressure distribution is non-uniform due to the curvature of streamlines and the pressure is lower at the crest and higher at the crown. Keeping these in mind, the total drop of siphon structure should be limited to about 6 m so that the negative pressures do not reach cavitation pressures.

To expedite the priming of siphon spillway, some kind of priming device is always used. The priming device could be a joggle (or step), a steel plate or some other suitable arrangement. A joggle, [Fig. 17.18 (a)] deflects the sheet of water flowing over the crest of the spillway to strike against the inner side of the hood thus forming a water seal which results in early priming of the spillway. The presence of the step, however, offers resistance to flow when the siphon ducts run full. A steel plate hinged at the spillway surface [Fig. 17.18 (b)] will also act as a priming device. Once the siphon duct starts running full, the plate is pressed downwards and is flush with the spillway surface so that there is no obstruction to the flow.



Siphon spillway with a joggle (b) Siphon spillway with a flexible steel plate

Fig. 17.18 Priming of siphon spillway

If the permissible negative head is h_0 and the radii of curvature of the crest and crown are r_0 and R_0, respectively, the unit discharge, q, through a siphon spillway, shown in Fig. 17.17, can be worked out on the assumption of free vortex conditions which are approximately obtained.

If v is the velocity of flow at radius r, then in a free vortex flow

$$vr = v_0 r_0$$

where, v_0 is the velocity at radius $r = r_0$ (i.e., the crest) and equals $\sqrt{2gh_0}$

$$\therefore \quad v = \frac{r_0}{r}\sqrt{2gh_0}$$

$$\text{Thus,} \quad q = \int_{r_0}^{R} v\,dr = r_0\sqrt{2gh_0}\int_{r_0}^{R}\frac{dr}{r}$$

$$\text{or} \quad q = r_0\sqrt{2gh_0}\,\ln\frac{R_0}{r_0} \tag{17.24}$$

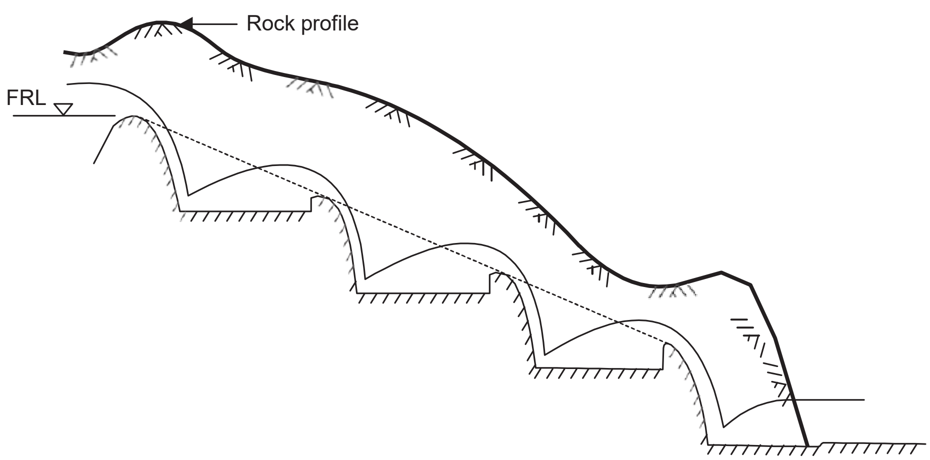
The main advantages of siphon spillway are: (i) its automatic operation without any mechanical device, and (ii) its ability to pass higher discharges at relatively low surcharge head resulting in lower height of dam as well as less surrounding area to be acquired for reservoir submergence.

Besides being an expensive structure and of limited capacity, it has a serious disadvantage due to the occurrence of sudden surges and stoppages of outflow as a result of erratic siphonic action, thus causing severe fluctuations in the downstream river stage. A minor crack in the cover of the siphon would interfere with the siphon. Therefore, siphon spillway is usually constructed in batteries so that the entire spillway is not affected even if cracks have developed in one or few units. In addition, the structure and foundation have to be strong enough to resist vibration stresses. Further, there exists a possibility of clogging of the siphon due to debris and floating material. Like other types of closed conduit spillway, a siphon spillway too is incapable of handling flows appreciably greater than the designed capacity. As such, siphon spillway, whenever provided, is used as a service spillway in conjunction with an auxiliary or

emergency spillway. In canyons of small width and small flood discharge, the suitability of a siphon spillway should always be examined.

Cascade Spillway

In case of very high dams the kinetic energy at the toe of the dam will be very high and the tail-water depth in the river may not be adequate for a single-fall hydraulic jump or roller bucket stilling basin. Narrow and curved canyons consisting of fractured rock would not be suitable for trajectory buckets. In such situations, especially for high earth and rockfill dams for which spillway is a major structure, possibility of providing a cascade of falls with a stilling basin at each fall (Fig. 17.19) must be considered. The cascade spillway (2, 8) is likely to be an ideal choice for a high rockfill dam for which the material has been obtained from a quarry located downstream of the dam so that the flood waters may be discharged over the quarry face. As the quarry would usually be excavated in benches, they may as well form the steps in the cascade. A cascade spillway has been planned at the proposed 218 m high Tehri dam on the Bhagirathi river in the Ganga valley of the central Himalayas. At Darmouth dam in Australia, the spillway to pass 2700 m^3/s of flood discharge is an unlined cascade in granite. The benches of the cascade will be 5 m high and of varying widths to suit the topography of the site. Although, a cascade spillway would be an attraction during floods, it may not be always acceptable for environmental reasons which demand that the quarries be always located upstream of the dam and below the normal water surface level of the reservoir so as to cause minimum disfigurement of the land.



River bed

Fig. 17.19 Cascade spillway

Tunnel Spillway

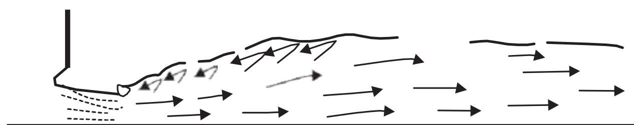
A tunnel spillway discharges water through closed channels or tunnels laid around or under a dam. The closed channels can be in the form of a vertical or inclined shaft, a conduit constructed in an open cut and back-filled with earth materials, or a horizontal tunnel through earth or rock. In narrow canyons with steep abutments as well as in wide valleys with abutments far away from the stream channel, tunnel spillways may prove to be advantageous. In such situations, the conduit of the spillway can be easily located under the dam near the stream bed.

TERMINAL STRUCTURES FOR SPILLWAYS

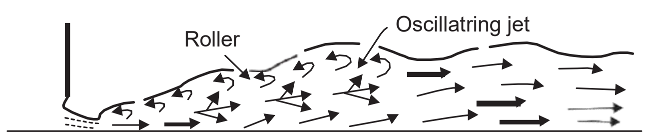
Some kind of energy dissipation is usually required before the spillway discharge is returned to the downstream river channel. An energy dissipator at the toe of the spillway is necessary to avoid or minimise erosion of river bed on the downstream side of the dam. A hydraulic jump is one of the best means to dissipate the excess energy of the falling water. Hence, wherever the tail-water conditions are suitable for the formation of a hydraulic jump, it is usual to provide hydraulic jump-type stilling basins which have been comprehensively studied by USBR (9). Alternatively, bucket type energy dissipators are provided.

Hydraulic Jump-Type Stilling Basins

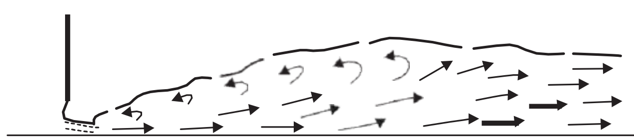
Selection of a suitable type of stilling basin depends upon the characteristics of the hydraulic jump that would form. The characteristics of the jump, in turn, depend on the Froude number of the incoming flow as has been illustrated in Fig. 17.20. For Froude numbers between 1.0 and 1.7, the depth of incoming flow is only slightly less than the critical depth. The change from supercritical stage to subcritical stage is gradual and the water surface is only slightly ruffled. For Froude numbers between 1.7 and 2.5, surface rollers are formed but the flow is still relatively smooth (form A of Fig. 17.20). For Froude numbers between 2.5 and 4.5, the incoming jet intermittently flows near the bottom and along the water surface and thus forms



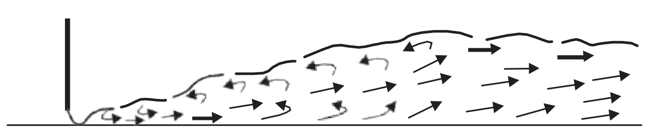
Form A - Pre - Jump stage ($1.7 < F_1 < 2.5$)



Form B - Transition stage ($2.5 < F_1 < 4.5$)



Form C - Range of well balanced jumps ($4.5 < F_1 < 9.0$)



Form D - Effective jump but rough surface downstream ($F_1 > 9.0$)

Fig. 17.20 Different forms of hydraulic jump

an oscillating hydraulic jump (form B of Fig. 17.20). The oscillating flow causes objectionable surface waves. A stable and well-balanced jump forms when the Froude number of the incoming flow lies between 4.5 and 9.0 (form C of Fig. 17.20). Water surface downstrem of the jump is relatively smooth and the action of the turbulence is confined within the body of the jump. When the Froude number exceeds 9.0, the surface roller and the turbulence are very active resulting in a rough water surface with strong surface waves downstream of the jump (form D of Fig. 17.20).

Of the different types of USBR stilling basins, Type III (Fig. 17.21) is commonly used. This basin is suitable when the Froude number of the incoming flow exceeds 4.5, and thevelocity of incoming flow does not exceed 15.0 m/s. The purpose of providing accessories, such as baffle blocks, chute blocks, and sill is to ensure the formation of the jump even in conditions of inadequate tail-water depth, and thus permit shortening of the basin length. Energy dissipation is due to the turbulence in the jump and also by the impact on blocks. Because of the large impact forces on the baffle blocks and owing to the possibility of cavitation along the

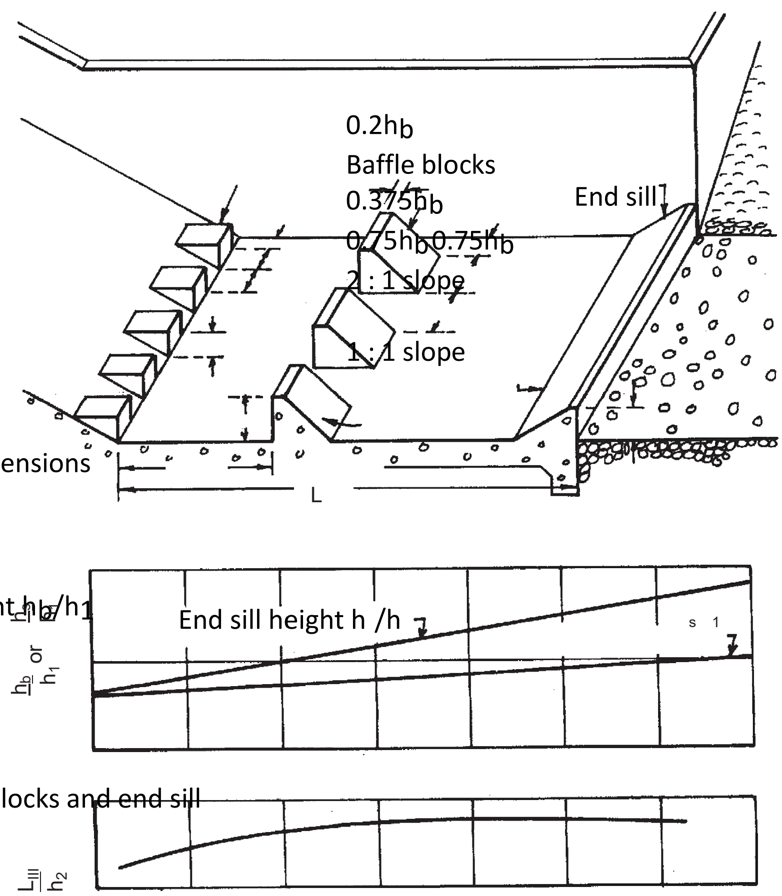
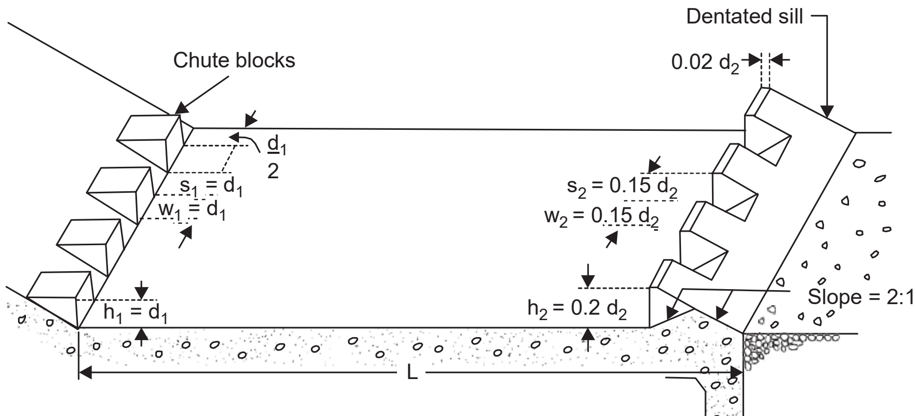


Fig. 17.21. USBR stilling basin Type III

surface of the blocks and floor, the use of this basin is limited to such conditions in which the velocity of incoming flow does not exceed 15.0 m/s. For good hydraulic performance, the side walls of a stilling basin are kept vertical or as nearly vertical as is practicable. For known conditions of incoming flow, the parameters of stilling basin can be determined from the curves of Fig. 17.21. A freeboard of 1.5 to 3.0 m should always be provided to allow for surging and wave action in the stilling basin.

When the velocity of the incoming flow exceeds 15 m/s, or when baffle blocks are not to be used, the USBR stilling basin, designated as Type II and shown in Fig. 17.22, should be adopted. Since the energy dissipation is accomplished mainly by hydraulic jump action, the basin length is bound to be longer than required for Type III basin. Also, the water depth in the basin should be about 5 per cent larger than the computed value of the post-jump conjugate depth.



(a) Type II basin dimensions

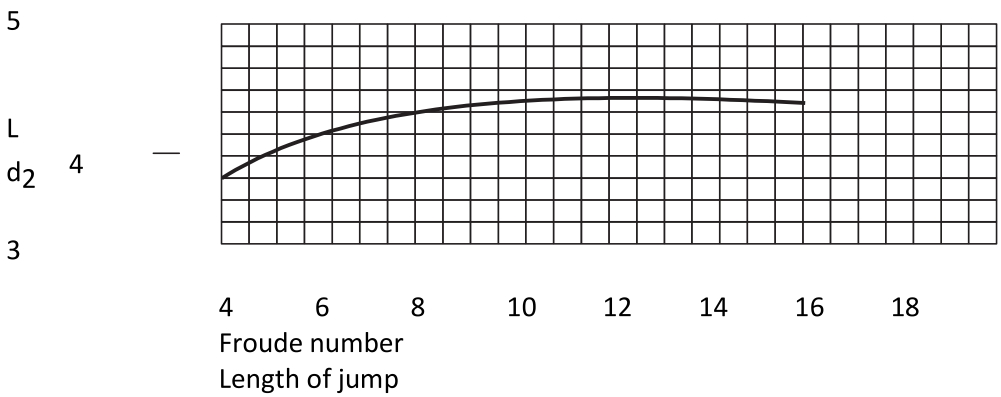


Fig. 17.22 USBR stilling basin Type II

Example 17.4 Design a stilling basin of USBR type III for an ogee spillway with the following data:

Design discharge = 13875 m^3/s Tail-water level at the design discharge = 180.30 m Crest length of spillway = 183 m

Tail-water depths (h_t) and post-jump depths (h_2) for different discharges are as follows:

Q (m^3/s)	0	3000	6000	9000	12000	13875
h_t (m)	0	6.90	9.40	11.30	12.90	13.80
h_2(m)	0	8.47	12.11	14.99	17.41	18.79

Solution : For the design discharge of 13875 m^3/s, h_2 = 18.79 m

$$F_2 = \frac{13875 / (183 \times 18.79)}{\sqrt{9.81 \times 18.79}} = 0.297$$

$$\therefore \quad h_1 = \frac{h_2}{2}\left[\sqrt{1 + 8F_2^2} - 1\right]$$

$$= \frac{18.79}{2}\left[\sqrt{1 + 8\,(0.297)^2} - 1\right] = 2.875 \text{ m}$$

$$F_1 = \frac{13875 / (183 \times 2.875)}{\sqrt{9.81 \times 2.875}} = 4.966$$

From Figs. 17.21 (b) and (c)

$$\frac{h_b}{h_1} = 1.49$$

$$\therefore \quad h_b = 4.28 \text{ m}$$

$$\frac{h_s}{h_1} = 1.30$$

$$\therefore \quad h_s = 3.74 \text{ m}$$

$$\frac{L_{III}}{h_2} = 2.32$$

$$\therefore \quad L_{III} = 43.59 \text{ m}$$

Other dimensions of the stilling basin as well as chute and baffle blocks can be determined using Fig. 17.21 (a).

Bucket-Type Energy Dissipators

When the tail-water depth is either too small or too large for the formation of hydraulic jump, the high amount of energy at the toe of spillway can be dissipated by the use of bucket-type energy dissipators. These can be either: (i) a trajectory (or deflector) bucket, or (ii) a roller (or submerged) bucket energy dissipator.

Trajectory (or Deflector) Bucket: When the tail-water depth is lesser than the depth required for the jump formation and the bed of the river channel is composed of sound rock capable of withstanding the impact of the trajectory jet, a trajectory bucket (also known as a flip or ski-jump bucket) (Fig. 17.23) is generally used as an energy dissipator. The incoming jet of water leaves the bucket as a free-discharging upturned jet and falls into the stream channel some distance downstream of the end of the spillway. The upturned jet gets split into a number of bubbles or smaller jets. The energy is dissipated on account of the increased air resistance (because of splitting of the jet) as well as the impact against water and the channel bed downstream.

With the end of the lip as the origin of the coordinate system, the path of the trajectory is given by the equation (10):

$$y = x \tan \alpha - \frac{x^2}{4k_1 E \cos^2 \alpha} \qquad (17.25)$$

where, α, known as the lip angle, is the angle between the curve of the bucket at the lip and the horizontal, E the specific energy, and k_1 is assigned a value of 0.85 to compensate for loss of energy and velocity reduction due to the effect of the air resistance, internal turbulence, anddisintegration of the jet.

Reservoir water surface

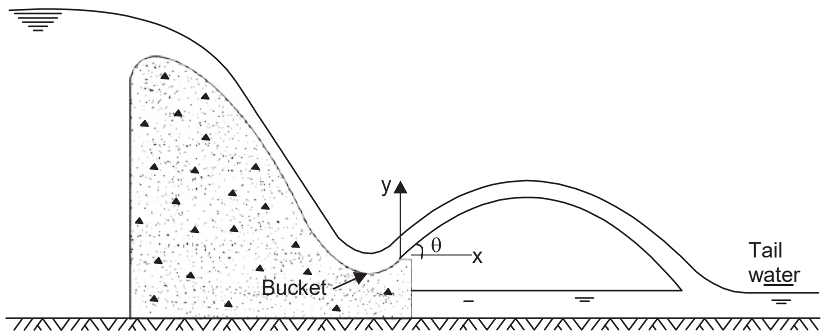


Fig. 17.23 Trajectory bucket

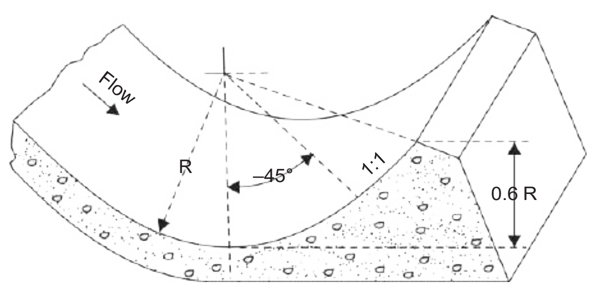
The horizontal range of the jet at the level of the lip (i.e., y = 0) is given by

$x = 4k_1E \tan \theta \cos^2 \theta$

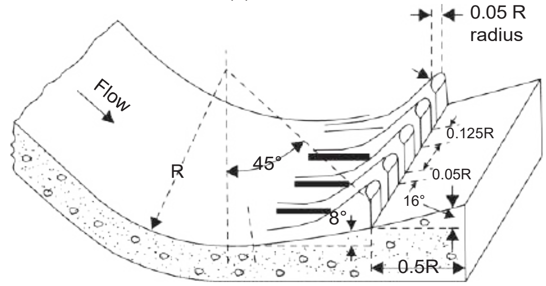
$= 2k_1E \sin 2\theta$

The maximum value of x will be 2 k_1E when θ is 45°. The lip angle depends on the radius of the bucket and the height of the lip above the bucket invert. It usually varies from 20° to 45°. The bucket radius should be large enough to ensure a concentric flow along the bucket (10).

Roller (or Submerged) Bucket: A roller bucket is used when the tail-water depth is greater than 1.1 times the required conjugate depth for the formation of the hydraulic jump and the river bed rock is good (10). Roller buckets can be either solid or slotted (Fig. 17.24). The



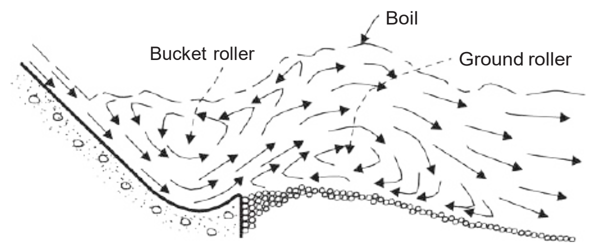
(a) Solid bucket



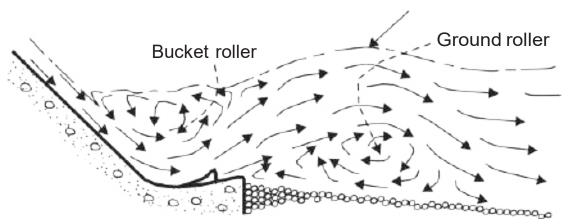
Slotted bucket

Fig. 17.24 Roller buckets

general hydraulic action responsible for the dissipation of energy is shown in Fig. 17.25. The energy dissipates through the formation of two rollers; one is on the surface of the bucket and moves anticlockwise (considering that the flow is to the right), and the other is a ground roller moving in a clockwise direction and which forms immediately downstream of the bucket (Fig. 17.25). The intermingling of the incoming flow with the roller as well as the movements of the latter dissipate the energy of the water effectively and prevent excessive scouring downstream of the bucket.



Solid bucket
Standing wave



Slotted bucket

Fig. 17.25 Roller formation in roller buckets

Although the hydraulic action of the two buckets is similar, there are some differing flow features as well. The deflector lip of a solid bucket directs upward the high-velocity flow and thus creates a high boil on the water surface and a violent ground roller moving clockwise immediately downstream of the bucket. This ground roller picks up the loose material and keeps some of it in a constant state of agitation. There may be unwanted abrasion on the concrete surfaces because of the loose material which is brought back towards the lip of the bucket by the ground roller. Further, the more violent water surface created by surface boil is carried downstream causing objectionable eddy currents which may adversely affect the river banks. In the slotted bucket, the high-velocity jet leaves the lip of the bucket at a relatively flatter angle and only a part of the high-velocity flow finds its way to the water surface. Thus, a relatively less violent surface boil forms and there is better dispersion of the flow in the region above the ground roller. Therefore, there is less concentration of high-energy flow throughout the bucket and a smoother downstream flow.

TYPES OF GATES FOR SPILLWAY CRESTS

Free or uncontrolled overflow crest of a spillway is the simplest form of control as it automatically releases water whenever the reservoir water level rises above the crest level. For such crests, there is no need of constant attendance and regulation of the control devices by an operator. Besides, the problems of maintenance and repair of the controlling device also do not arise. However, when sufficiently long uncontrolled crest or large surcharge head for the required

spillway capacity cannot be obtained, a regulation gate may be necessary. Such regulating devices enable the spillway to release storages even when the water level in the reservoir is below the normal reservoir water surface. Gates can be provided on all types of spillways except the siphon spillway. The installation of gates involves additional expenditure on initial cost, and on their repair and maintenance. The selection of type and size of the controlling device depends on several factors, such as: (i) discharge characteristics of the device, (ii) climate,
frequency and nature of floods, (iv) winter storage requirements, (v) the need for handling ice and debris, and (vi) special operating requirements such as presence of operator during periods of flood, the availability of electricity, operating mechanism, and so on. In addition, economy, reliability, efficiency, and adaptability of the regulating device must also the looked into.

The following types of regulating devices are generally used:

Flashboards and stoplogs,

Rectangular lift gates,

Radial gates, and

Drum gates.

These may be controlled either manually or automatically through mechanical or hydraulic operations.

Flashboards and Stoplogs

Flashboards and stoplogs raise the reservoir storage level above a fixed spillway crest level when the spillway is not required to release flood. The flashboards usually consist of individual boards or panels of 1.0 to 1.25 m height. These are hinged at the bottom and are supported against water pressure by struts. Stoplogs are individual beams or girders set one upon the other to form a bulkhead supported in grooves at each end of the span. To increase the spillway capacity, the flashboards or stoplogs are removed prior to the flood. Alternatively, they are designed and arranged so that they can be removed while being overtopped. Flashboards and stoplogs are simple and economical type of regulating devices which provide an unobstructed crest when removed. However, they have the following disadvantages (1):

They present a hazard if not removed in time to pass floods, especially where the reservoir area is small and the stream is subject to flash floods,

They require attendance of an operator or crew to remove them, unless designed to fall automatically,

Ordinarily they cannot be restored to position while water flows over the crest,

If they are designed to fail when the water reaches certain stage, their operation is uncertain, and when they fail they release sudden and undesirably large outflows, and

If the spillway functions frequently, the repeated replacement of flashboards may be costly.

Vertical Lift Gates

These are usually rectangular in shape and made of steel which span horizontally between guide grooves in supporting piers and move vertically in their own plane. The gates are raised or lowered by an overhead hoist and water is released by undershot orifice flow for all gate openings. Sliding gates offer large sliding friction due to water pressure and, therefore, require a large hoisting capacity. The use of wheels (along each side of the gate) would reduce the amount of sliding friction and thereby permit the use of a smaller hoist. Vertical lift gates have been used for spans and heights of the order of 20 m and 15 m, respectively. At larger heights, however, the problem of a raised operating platform becomes important.

Radial (or Tainter) Gates

These are made of steel plates which form a segment of a cylinder which itself is attached to supporting bearing by radial arms. The cylindrical plate is kept concentric to the supporting pins so that the entire thrust of the waterload passes through the pins and only a small amount of moment needs to be overcome in raising or lowering the gate. The hoisting loads then include only the weight of the gate, the sliding friction, and the frictional resistance at the pins. The small hoisting effort required for the operation of the radial gates makes hand operations at small installations possible. Besides, they require lesser head rooms than required by vertical lift gates. All these advantages make the radial gates more adaptable.

Drum Gates

Drum gates are hollow (and, therefore, buoyant), triangular in section, and made of steel plates. The drum gate is hinged at the upstream lip of a hydraulic chamber in the weir structure in which the gate floats. Water introduced into or drawn from the hydraulic chamber causes the gate to swing upwards or downwards. The inflow or outflow of water to the chamber is governed by controls located in the piers adjacent to the chambers.

CAVITATION EROSION ON SPILLWAY SURFACE

Surface roughnesses of a spillway causes separation of the boundary layer from the surface, thereby forming a lower pressure zone immediately downstream of the roughness. If the pressure in the wake region of the roughness falls to the vapour pressure of water, vapour bubbles are formed and carried downstream. When these bubbles reach a high pressure region, they suddenly collapse giving rise to extremely high pressures. These extremely high localised pressure cause damage to any surface adjacent to the collapsing bubbles (or cavities). The continued process of this nature, named as cavitation pitting or cavitation erosion, may cause serious damage to the structure in due course of time.

With the increasing demand for water, high dams are being proposed, planned, and constructed. The flow velocities over the spillways of such projects often exceed 30-40 m/s which are enough to cause cavitation pitting of the normal concrete surface of the spillway. The methods to prevent cavitation, used till recently, consisted of either using cavitation- resistant materials (such as steel lining, epoxy concretes, epoxy mortars, and fibrous concrete) for the construction of the surface or the adoption of specification criteria for limits of construction finish. The first method, being costly, is reserved for small areas such as near the outlet gates, or for repairing damaged surfaces. The second method defines objectionable irregularities. However, the standards are difficult to obtain especially when the flow velocities exceed 20 m/s. Besides, the method is relatively uncertain because of the defects which may develop on the surface as a result of climatic, atmospheric or chemical conditions.

More recently, a new method has been developed to protect spillway surfaces by aeration devices. This method is based on the known fact that the presence of air bubbles hinders cavitation. Hence, present-day design of spillways with high velocities envisages provision of aeration devices, known as spillway aerators, across the spillway face. Three basic types of aerators, viz., groove, deflector, and offset (Fig. 17.26) have been used. An aerator of the groove- type has been constructed for the Karjan dam in the Narmada basin.

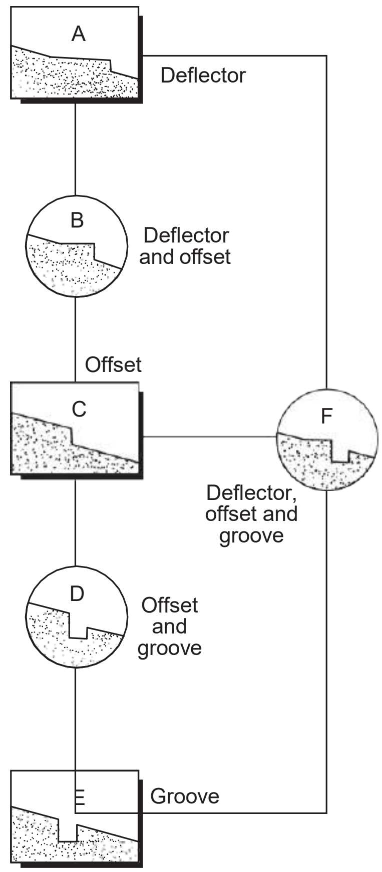


Fig. 17.26 Spillway aerators

The aerators are relatively cheap and have proved successful in preventing cavitation. These devices produce a local pressure drop which causes the flow to suck air. The compressibility of the air-water mixture reduces considerably the collapsing pressure of vapour bubbles and thus protects the concrete surface from cavitation erosion.

Negative Impact of Multipurpose Project or Climate Change Impacts in Water Resources

The water resources system is very sensitive to climate change. Climate change has an impact on such important natural phenomena as precipitation, flood and the water supply in rivers. They, in turn, directly or indirectly affect factors as fundamental as biodiversity, food production, and health and power generation. The assessment of these impacts is very essential for an efficient, effective and sustainable planning of water resources. Besides, water is getting increasingly scarce. Hence, the planners and managers concerned need to be aware of the impacts of climate change on water resources.

Precipitation Change

Impacts on Water Availability

Impacts on Flood and Drought

Impact on Agriculture and Livelihoods

Impact on Hydropower

Perceptions of the People on the Livelihood Systems in the High Himalaya

Impact on agriculture

Winter snowfall is the main source of moisture supply and irrigation for agriculture during the cropping season. Less amount of snowfall in winter will affect the crops. Heavy snowfall during winter is always preferred because of the prolonged supply of moisture to the crops during the summer. However, it may also lead to the casualty of lives and livestock.

Late snowfall or snowfall during spring affects the cropping schedule. Recently, such changes in the seasonal pattern have inhibited the farmers to sow crops like potato in prime time. This has resulted in the emergence of some pests and diseases that affect both the plant and its harvesting.

Forward shift in the cropping schedule is likely to have a negative impact on tourism in the area. The late harvest would encroach upon the tourist season (mid-September to December) which is a crucial source of cash income in the area.

Rapid melting of snow and as a result, of snow ice would cause huge amounts of debris and sediment delivery which may destroy their agricultural lands, forests and pastures.

The present trend of warming will perhaps favour agriculture in the long run by providing scope for the cultivation of crops adapted to warmer conditions.

Impact on livestock

Decline in snowfall in the winter is likely to reduce the availability of pastures in the area during summer. This in turn is likely to result in downsized livestock numbers and may reduce production. It has to be reckoned that alpine pasture-based livestock rearing is still an integral component of the livelihood system in the area.

There is a repercussion of pasture decline in crop cultivation. The decrease in grazing livestock will lead to the reduction of manure supply for the farms as well as the grazing areas. This in turn will inhibit the recovery of both the crop fertility and the pastures which ultimately is likely to hamper the regenerative capacity of the production system.

Livestock has, in the past, played a key role in the transportation of goods across the trans-Himalaya. Even today, it forms a crucial part of the trekking industry. Jokpio (a cattle-yak cross breed), an animal used for transportation, is either the main or subsidiary source of income for many households. But, it is likely to decrease in numbers if enough pastures are not available.

Impact on fresh water availability

Freshwater is life supporting system. People perceive that the decline in the snow mass in the Sagarmatha region is likely to reduce the availability of freshwater. Freshwater - apart from a source of moisture supply to agro-pastoralism - is used for drinking, cooking, washing, wine making, hotels and lodges, irrigation, running of ghatta (hydropowered local grain mills), maaney (prayer wheel) and micro hydropower including peltriset, and for religious purpose like pilgrimages.

Although climate change is a global phenomenon, both its trends and impacts may be different on a local scale. The local hydrology of every river in the world is likely to be affected by climate change in some ways. It affects different aspects of the local hydrology of a river such as the timing of water availability, quantity and quality. These changes in the river hydrology will induce risks to water resources facilities in the form of flooding, landslides, sedimentation from more intense precipitation events (particularly during the monsoon) and greater unreliability of dry season flows.

The latter poses potentially serious risks to water and energy supplies in the lean season. For the long-term planning and management of water resources, the future changes in the pattern of land use, water demand and water availability should be analyzed well in advance. It entails understanding the manner in which a water resources system responds to changing trends and variability, the manner in which it is affected by these conditions today and how it might respond if these conditions undergo change. The assessment of climate change helps build resilience against its possible impacts through enhanced institutional flexibility and the consideration of climate-related risks in the planning process.

The impact of climate change on water resources depends not only on changes in the volume, timing, quality of the stream flow and the recharge but also on system characteristics, changing pressures on the system, the manner in which the management of the system evolves and the adaptation measures implemented for climate change. Non-climate changes may have a greater impact on water resources than climate change in the managed basins. But, the unmanaged systems are likely to be most vulnerable to climate change. Climate change challenges the existing water resources management practices by adding an additional uncertainty. With the increasing variability in climate which is sure to grow even more extreme in the future, developing countries are particularly vulnerable to extreme weather events.

Hydrologically, water resources in Nepal can be categorized into three different groups: (a) dry pre-monsoon season (March–May) with almost no rain (b) rainy monsoon season (June–September) (c) post-monsoon season (October–February) with little rain. Most of the agricultural water demands are by rice in the monsoon, wheat in the post-monsoon and by maize in the pre-monsoon season. The following are the features of river flow in Nepal

Less monsoonal rains across the high mountains and more monsoonal rains along the southern hills.

The frequency and duration of small floods affected the most. Although their magnitude is decreasing, floods in the river seem to be more frequent and of longer duration.

Snow fed rivers have an early shifting and non-snow fed rivers have a late shifting of the hydrograph.

Rapid retarding of glaciers. The formations of glacier and supra-glacier lakes pose great threats of GLOF to infrastructure downstream.

Rapid decrease of snow cover will firstly diminish the storage capacity of natural reservoirs and then the dry season flow in the rivers; and Climate change will produce excess water in the wet season but little flow in the dry season. This will pose a challenge to the planners and managers concerned.

The poor are more vulnerable to climate extremes as well as gradual changes in the climate as they have less production, less reserves, fewer alternatives and lower adaptive capacity. About 31% of the population in Nepal is below the poverty line and 95% of them live in rural areas. Moreover, as much as 70% of the rural population is poor and the local food production sometimes just covers three months of the annual household needs. The impacts of climate change on the livelihoods of the poorest of the poor in Nepal would therefore be substantial.

Climate change and climate risks in general are neglected in the country's development policy. For example, power sector plans do not recognize the risks faced by the hydropower plants due to the variability in the runoff, floods (including GLOFs) and sedimentation. Similarly, climate risks have not been mentioned in the irrigation sector plan. However, some of the activities like the mitigation of floods, mitigation in the erosion of cultivated areas and water harvesting to provide year-round water supply for irrigation would fit well into an adaptation strategy for Nepali agriculture. Likewise, the introduction of a non-conventional irrigation project in the Department of Irrigation also fits well into adaptation measures.

Through this project, the department has already implemented sprinkler and drip irrigation systems in 1800 ha of land and has set a target for 10,000 ha. The introduction of the poor and the marginalized to this non-conventional system would strengthen their coping capacity. This would be brought about with the introduction of diversified crops and less dependency on rain-fed conventional agriculture and the diminishing supply from surface irrigation, both of which are highly vulnerable to climate change.

For sites lacking perennial or adequate water sources, simple solutions like rainwater harvesting schemes and solar pumps are also included in the government policy. However, the real climate-related risks (what is "adequate" and how do you deal with a water source that is usually perennial but dries up during a period of drought) are not discussed.

Activities like the establishment of a national disaster preparedness and management agency, the creation of village-level early warning systems for floods, landslides etc., building decentralized emergency response capacity, enforcing design standards for buildings and infrastructure that take into account site-specific risks, investing in better weather and earthquake prediction systems, the monitoring of lakes and preparation of siphon materials(specifically for

GLOFs) are some of the coping measures adopted by the Nepal government in the context of climate change.

One adaptive response to GLOF risks is to promote the development of smaller hydropower plants. This would spread the risk in the event of a catastrophic flood and negate the scenario of damage to a huge plant with significant sunk costs. The development of micro and small hydro is already in line with Nepal's development priorities and is being encouraged by both the government and the donors. In other words, climate change might be one supplementary motive to promote a strategy that is already being implemented for reasons of economic development.

The introduction of multiple units in power plants, alternative sources of energy supply and a better demand-side management are some of the noted approaches adopted by Nepal in coping with the adverse effects of climate change in the hydropower sector. In addition, the initiation of Optimum Sediment exclusion (OSE) research in the Jhimruk and Khimti hydropower plants is a step forward in the adaptation/mitigation measure in the context of climate change. It will help improve the performance of both the existing as well as the planned hydropower projects. This will lead to the maximizing of benefits from such projects and the minimization of both the construction costs and the overall environmental effects caused by the construction of new projects to meet an equivalent energy demand.

In Nepal, Kulekhani is the only storage project generating electricity. By the efficient utilization of RoR hydropower plants, equivalent water can be saved in the Kulekhani reservoir to generate power at peak hours. This can help in reducing the duration of the present load-shedding hours. As an example, a 2% increase in the performance of the existing power plants in Nepal can generate the equivalent of 11 MW of extra power which is equivalent to 264,000 units per day. In the dry season, an equivalent amount of energy in the form of water can be stored in the Kulekhani reservoir and supplied during the peak hours.

River-linking projects like the Sunkoshi-Kamala diversion, the Bheri-Babai diversions and several multipurpose high dam projects are envisaged in the government policy. They act as additional intentions in place to cope with the stream flow fluctuations as a result of the current seasonal as well as climate variability.

There is an urgent need for the optimal use of available water in order to increase agricultural production. Out of the total cultivable area of 2.64 million hectares in Nepal, only 1.14 million hectares (i.e. 43%) had modern irrigation facilities at the end of 2004 (MoF, 2005). The productivity of the irrigated areas in Nepal is one of the lowest in South Asia. There are a number of factors behind this low level of production but reliable irrigation is the key issue. Reliability is a relative term and a scheme supposed to be reliable twenty years ago may not be rated so now. To meet the demand for more food, the productivity has to be increased. At the same time, high valued crops have to be grown in order to make agriculture profitable. These demand for a more flexible and assured water supply that is difficult with the present level of infrastructure and the design and operation concepts.

Way Forward, Future Activities

In order to address the issues related to climate change, it is necessary to strengthen and expand the hydrological and meteorological observation networks for short-, medium and long -term data collection. The station network should be as per the WMO standard. Likewise, there should be adequate instruments for data analysis and communication as per the need of the project and a proper mechanism to assure the long-term sustainability of the observation network. The network should be equipped with robust tools, effective methods and capable human resources for data collection, compilation, and processing, monitoring, evaluation and upgrading.

Similarly, the establishment of a strong water resources database including snow and glacier information is required. Integrated river basin study, pilot river basin study demonstration projects, development of suitable climate models, establishment of a model test laboratory, redefinition of the water structure design criteria and identification of vulnerable areas and climate friendly technologies, development of adaptive measures and proper implementation, researches on water resources for the application of the 3R (reduce, reuse and recycle) principle as well as studies on addressing the climate change impacts on landslides, debris flows, floods and droughts are also necessary to cope with climate change. The creation of mass awareness, formation of climate change sectoral policies on matters related to water resources, stringent judicial enforcement, reliable information of the Himalayan snowfields and glaciers, capable human resources development and the creation of opportunities for higher studies are some of the other necessities that are to be planned and carried out in a proper and effective manner.

Climate change impacts are observed in several sectors of Nepal among which water resources is one of the hardest hit sector. Evaluating the impacts in water resources is challenging because water availability, quality and stream flow are sensitive to changes in temperature and precipitation. Increased demand for water caused by population growth, changes in the economy, development of new technologies, changes in watershed characteristics and water management decisions are some of the other factors to be taken into consideration. Water is considered to be a vehicle to climate change impacts and hence needs to be handled carefully and skillfully.

Nepal is rich in water resources. There are about 6000 rivers in Nepal having drainage area of 191000 sq. km, 74 % of which lies in Nepal alone. There are 33 rivers having their drainage areas exceeding 1000 sq. km. Drainage density expressing the closeness of spacing of channels is about 0.3 km/sq. km. If this natural resource is properly harnessed, it could generate hydropower; provide water for irrigation, industrial uses and supply water for domestic purposes.

Rivers of Nepal can be broadly classified into three types, in accordance to their origins: The fi rst category comprises of the four main river systems of the country: Koshi, Gandaki, Karnali and Mahakali river systems, all of them originating from glaciers and snow-fed lakes. Rivers of the second category originate from Mahabharat range which includes Babai, West Rapti, Bagmati, Kamala, Kankai and Mechi etc. Streams and rivulets originating mostly from the Chure hills make up the third category; these rivers cause fl ash floods during monsoon rains and remain without any flow or very little flow during the dry season.

Currently, about 10% of total precipitation in Nepal falls as snow, about 23% of Nepal's total area lie above the permanent snowline of 5000 m., about 3.6% of Nepal's total areas are covered by glaciers. There are 3,252 glaciers covering an area of 5,323 sq.km with an estimated ice reserve of 481 km3 . There are 2323 glacial lakes in Nepal covering an area of 75 sq.km.

The surface water available in the country is estimated to be about 225 billion m3 (BCM) per annum or equivalent to an average fl ow of 7,125 m3 /s, out of which only 15 BCM per annum is in use. Around 95.9% of 15 BCM has been used for agriculture, 3.8% for domestic purpose and only about 0.3% for industry. It is observed that around 78% of the average flow of the country is available in the first category river basins, 9 % in the second category basins and 13 % in the numerous small southern rivers of the Terai. Studies have shown that the first Category Rivers have surplus flow but the second category rivers have deficit flow in the dry season.

Nepal's economy is largely based on agriculture; it contributes about 40% to GDP and provides employment to two-thirds of the population. However, Nepalese agriculture is mainly rain fed and agriculture production in both rain fed as well as irrigated areas are being badly affected due to droughts, flooding, erratic rainfall, and other extreme weather events. Nepal was self sufficient in food grain production until 1990. Due to drought condition in 2005/06, production fell short by 21553 metric tonnes and by 179910 metric tonnes in 2006/07 due to drought and natural calamities.

Nepal has a cultivated area of 2,642,000 ha (18% of its land area), of which two third (1,766,000 ha) is potentially irrigable. At present 42% of the cultivated area has irrigation of some sort, but only 17% of cultivated area has year round irrigation. An estimate shows that less than 8% of the country's water potential is used for irrigation.

In addition to surface water, a large volume of water is available in the shallow and deep aquifers which are estimated to be 8.8 BCM annually which can be used for irrigation and domestic water supplies.

The estimated hydropower potential of Nepal is 83,000 MW of which 114 projects having 45,610 MW have been identified economically feasible. However in the context of climate change the hydropower development scenario needs to be revisited in totality. At present, Nepal Electricity Authority (NEA) has a total installed electricity generation capacity of about 689 MW, of which the hydropower capacity is 632 MW.

Only about 72% of the country's population has access to basic water supply and only 25% of the whole population has sanitation facility.

Nepal is highly vulnerable to recurrent floods and landslides. In Nepal, devastating floods are triggered by different mechanisms such as: i) continuous rainfall and cloudburst (CLOFs), ii) glacial lake outburst floods (GLOFs), iii) landslide dam outburst floods (LDOFs), iv) floods triggered by the failure of infrastructure, and v) sheet flooding or inundation in lowland areas due to an obstruction imposed against the flow.

According to the precipitation trend analysis, the annual average precipitation over Nepal is decreasing at the rate of 9.8 mm/decade, however the Koshi basin shows increasing trend. Trend of the annual discharge of three major River basins Koshi, Gandaki and Karnali indicates that the discharges in these major basins are decreasing annually but, the annual discharges in southern basins were in increasing trend.

Analyses of monthly flow trend of some of the rivers indicate that the contribution of snow melt in runoff is in increasing trend for snow-fed rivers, similarly for non snow-fed rivers, dry season flows are decreasing and wet season flows are increasing. It is also observed that the numbers of flood events are increasing as well as the effect of single flood is also increasing to more days. The changing precipitation pattern indicated that the drought period was becoming longer, though there was no definite trend in the annual precipitation amount.

The impact on snow and glacier is found to be very high. Negative trends are observed in the glacier mass balance. Glacial Lakes are expanding and the threats of Glacial lake Outburst Floods (GLOF) are ever increasing.

Agriculture is the mainstay of Nepal's economy. Climate variability directly affects agricultural production, as agriculture is one of the most vulnerable sectors to the risks and impacts of global climate change and water shortages. Any further decreases in water resources, especially during the non-monsoon seasons, would adversely affect agricultural production. It will have a direct impact on the livelihood of the people.

Climate change impacts on water resources may be addressed by focusing on i) Research, ii) Optimum observation network, iii) Strong data-base and, iv) research based action oriented program/projects. In general they are: establishment of a strong water-resources data base including snow and glacier information. Integrated river basin study, pilot river basin study demonstration projects, development of suitable climate models, establishment of a model test laboratory, redefinition of the water structure design criteria and identification of vulnerable areas and climate friendly technologies, development of adaptive measures and proper implementation, research on water resources to apply 3R (reduce, reuse and recycle) principle as well as study on addressing the climate change impacts on landslides, debris flow, floods and droughts are also necessary to cope with the climate change. Creation of mass awareness, formation of climate change sectoral policies on water resources relation matters, stringent judicial enforcement, reliable information on the Himalayan snow and glacier field, capable human resources development, creation of opportunities for higher studies are some other necessary things to be planned and carried out in a proper and effective ways.

Water and Energy Commission Secretariat (WECS) has to take the supervisory role in collection, compilation, processing and maintaining a strong data base and has to take a lead role in conducting the above activities.

The surface water available in the country is estimated to be about 22
annum, equivalent to an average flow of 7,125 m3/s (WECS, 2003). The t
of these rivers is around 194,471 km2, 76% of which lies within Nepal.

S.N.	River	Length (km)	Drainage Area (km2)		Estimat (m3
			Total	Nepal	From al Basins
1	Mahakali	223	15,260	5,410	698
2	Karnali	507	44,000	41,890	1441
3	Babai	190	3,400	3400	103
4	West Rapti	257	6,500	6,500	224
5	Narayani	332	34,960	28090	1753
6	Bagmati	163	3,700	3,700	178
7	Sapta Koshi	513	60,400	31940	1658
8	Kankai	108	1330	1330	68
9	Other River		24921	24921	1001
Total			194,471	147,181	7125

Table 1: Estimated runoff from the rivers of Nepal (WECS, 2003)

S.N.	Description	Unit	Estimated V
			For all Basins

Negative impacts of inter-basin water transfer

Inter-basin water transfers have been criticized by environmental organizations for several reasons. This is because; the development of inter-basin transfers has the potential to disturb the water balance in both the donating and the receiving region. In the past, certain inter-basin transfers have caused a disproportionate amount of damage to freshwater ecosystems in relation to the schemes' benefits. Negative social as well as economic impacts, especially for the donor basin, can also occur. Inter-basin transfers may not be the most cost effective way of meeting water demand in the receiving region. Furthermore, inter-basin transfers do not encourage users in the receiving region to use the water more effectively, to recycle wastewater or to develop new local water sources for supply. According to WWF (2007), the following negative impacts can be observed in certain cases of inter-basin water transfers:

Demand management in recipient basin is not sufficiently considered in preplanning for inter-basin transfer, leading to ongoing water waste.

Inter-basin transfers can become drivers for unsustainable water use in recipient's basin-irrigation and urban water use, and create strong dependence on inter-basin transfer in the recipient community.

The proliferation of boreholes to access groundwater can lead to overexploitation of this resource, too.

Inter-basin transfers can become a catalyst for social conflict between donor and recipient basins or with government

Inter-basin transfers may not help the situation of the poor affected or displaced by it.

Governance arrangements for inter-basin transfers can be rather weak, resulting in budget blow-out or corruption Source: WWF (2007)

3.4 RIVERS AND RIVER TRAINING METHODS

GENERAL

Rivers have always played an important role in human development and in shaping civilizations. Primary function of a river is the conveyance of water and sediment. Besides serving as asource of water supply for domestic, irrigation, and industrial consumption, rivers have been useful in providing facilities for navigation, recreation, hydropower generation, and waste disposal. Rivers, except when flowing through well-defined narrow sections confined by high and stiff banks, have also generally caused problems of flooding; change of course, banks erosion etc.

The structure and form of rivers including plan-forms, channel geometry (*i.e.,* cross-sectional shape of river), bed form, and profile characteristics together form what is termed river morphology. The morphology of river changes considerably on account of natural causes. Besides, changes made by man in an attempt to harness a river strongly influences behaviour of the river.

CLASSIFICATION OF RIVERS

Rivers can be classified as follows:

(i) Based on variation of discharge in river, as
- (a) Perennial rivers,
- (b) Non-perennial rivers,
- (c) Flashy rivers, and
- (d) Virgin rivers.

(ii) Based on stability of river, as
- (a) Stable rivers,
- (b) Aggrading rivers, and
- (c) Degrading rivers.

(iii) Based on the location of reach of river, as
- (a) Mountainous rivers,
- (b) Rivers in flood plains,
- (c) Delta rivers, and
- (d) Tidal river

(iv) Based on the plan-form of river, as
- (a) Straight rivers,
- (b) Meandering rivers, and
- (c) Braided rivers.

Perennial Rivers

Perennial rivers obtain their water from melting snow for the larger part of any year besides getting rain water during the rainy season. Being snow-fed, perennial rivers carry significant flow all through the year.

Non-perennial Rivers

Non-perennial rivers are not snow-fed rivers and, hence, get completely dried up or carry insignificant flow during the summer season. They get their supplies only during the monsoon as a result of rains in their catchment areas.

Flashy Rivers

In case of flashy rivers, the river stage rises and falls in a very short period of a day or two due to the steep flood hydrograph. A small flow may, however, continue for some time.

Virgin Rivers

In arid regions, waters of some rivers may get completely lost due to evaporation and percolation. Such rivers become completely dry much before they join another river or sea, and are called virgin rivers.

Stable Rivers

When the alignment of a river channel, river slope, and river regime are relatively stable and show little variation from year to year except that the river may migrate within its permanent banks (*i.e., khadirs*) (Fig. 12.1), the river is said to be stable. However, changes in bed and plan-forms of a stable river do take place, but these are small.

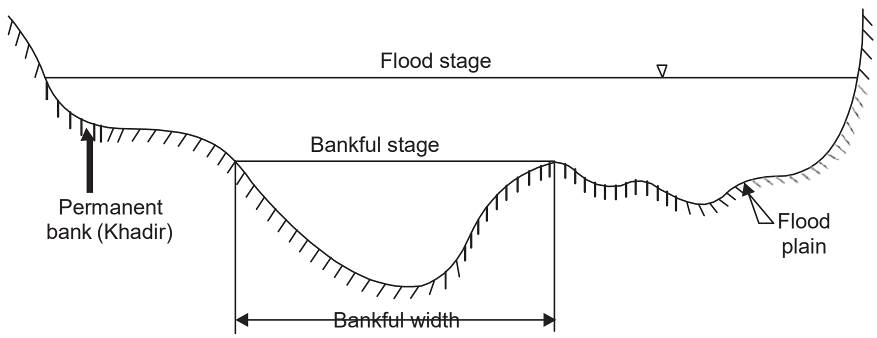


Fig. 12.1 Typical cross-section of river in flood plains

Aggrading Rivers

When the sediment load entering a river reach is greater than the sediment load leaving a river reach, the river in that reach becomes an aggrading river due to deposition of excess sediment. This situation may arise due to obstructions (*e.g.,* barrage or dam) across a river, extension of delta at the river mouth, or sudden intrusion of sediment from a tributary.

Aggrading rivers usually have straight and wide reaches with shoals in the middle which shift with floods. The flow in the river channel gets divided into a number of braided channels.

Degrading Rivers

When the sediment load entering a river reach is less than that leaving the river reach, the river in that reach becomes a degrading river due to erosion of the bed and bank material.

Mountainous Rivers

Rivers in mountainous reaches are further divided into incised rivers and boulder rivers. Incised rivers have a steep bed slope and high velocity of flow. The bed and the banks of these rivers are made up of rocks and very large boulders which are, usually, highly resistant to erosion. The sediment transported by an incised river is often different from that of the river bed and comes from the catchment due to soil erosion.

The bed and sides of a boulder river consist of a mixture of boulder, gravel, shingle, and sand. The bed slope and the velocity of flow are smaller than those of incised rivers. The river cross-section is usually well-defined. There is, however, considerable subsoil flow due to high permeability of the bed material.

Rivers in Flood Plains

After the boulder stage, a river enters the alluvial plains. The bed and banks are now made up of sand and silt. The bed slope and the velocity of flow in the river are much smaller than those of boulder rivers. The cross-section of the river is decided by the sediment load and the erodibility of the bed and banks of the river. A typical cross-section of a river with a flood plain is shown in Fig. 12.1. The sediment transported by such rivers is predominantly of the same type as the material forming the channel bed. During high floods, these rivers inundate very large areas and cause considerable damage to life, property, and crops. Such rivers are also called *alluvial rivers.*

Tidal Rivers

All rivers ultimately meet the sea. In the reach of a river just upstream of the sea, there would occur periodic changes in water levels due to tides. This reach of the river is called tidal river and receives sea water during flood tides and raises its level. During ebb tides, the river water level is lowered. The length of the river reach affected by tidal effects depends on the river slope, the tidal range, discharge, river configuration *etc.*

Delta Rivers

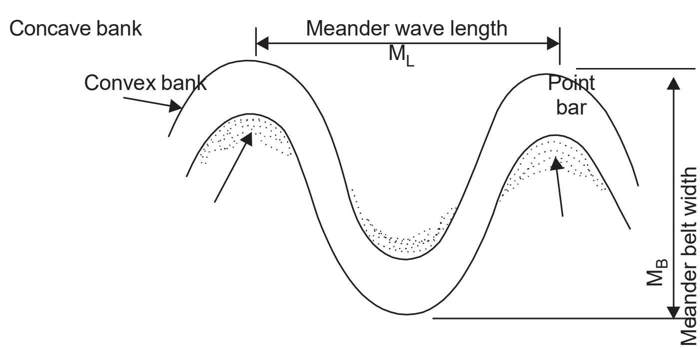
A river, before becoming a tidal river, may split into number of branches due to very flat bed slopes resulting in shoal formation and braiding of the channel. This part of the river reach is called delta river. The delta river indicates a stage, rather than a type of river.

Straight Rivers

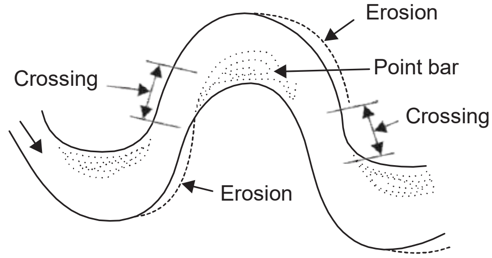
In the straight reaches of a river, its section has the shape of a trough and maximum velocity of flow occurs in the middle of the section. It is very difficult to find the straight reach of an alluvial river over large lengths. Alluvial rivers seldom run straight through a distance greater than ten times the river width (1). Even in the apparent straight reaches, the line of maximum depth - commonly known as *talweg* – moves back and forth from one *khadir* (permanent bank) to another *khadir*.

Meandering Rivers

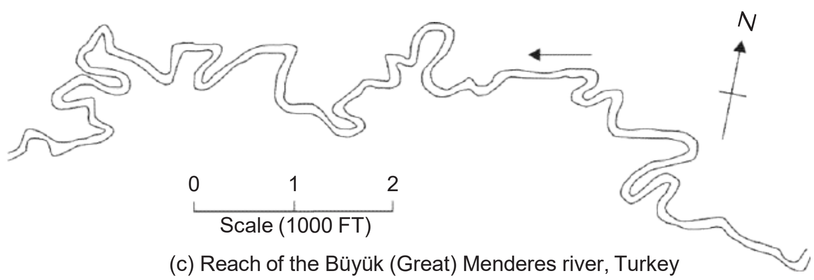
On account of the slight asymmetry of flow in alluvial rivers, there is a tendency for such rivers to vary their plan-forms into bends which eventually result in a meandering pattern (Fig. 12.2). The term meandering has been derived from the Great Menderes river in Turkey which follows a winding or intricate course (Fig. 12.2). Rivers having such meandering patterns are known as meandering rivers which, in plan, comprise a series of bends of alternate curvature. The successive curves are connected through straight reaches of the river called 'crossing'. Meandering increases the length of river and decreases its slope.



(a) Definition sketch



(b) Pattern of erosion and deposition



(c) Reach of the Büyük (Great) Menderes river, Turkey

Fig. 12.2. Meandering channel

Braided River

When a river flows in two or more channels around alluvial islands, it is called a braided river (Fig. 12.3). The braided patterns in a river develop after local deposition of coarser material which cannot be transported under prevailing conditions of flow and which subsequently grows into an island consisting of coarse as well as fine material.

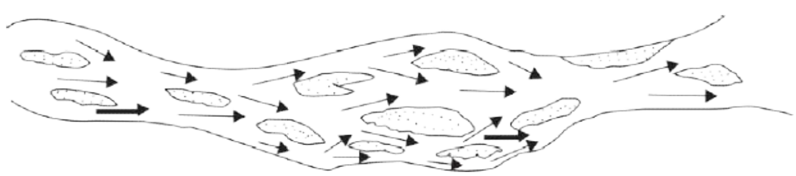


Fig. 12.3 Typical braided reach of a river

BEHAVIOUR OF RIVERS

The behaviour of a river is mainly affected by the characteristics of the sediment-laden water flowing in the river. The available energy of the flow is utilised in transporting the sediment load as well as in overcoming the resistance due to the viscous action and the roughness of bed and sides. On account of the interdependence of the factors affecting the flows, there is an inherent tendency of these rivers to attain equilibrium. As such, whenever the equilibrium of a river is disturbed by man-made structures or natural causes, the river tends to attain a new equilibrium condition by scouring the bed or by depositing the sediment on the bed or by changing its own plan-form. These changes can be either local or extended over a long reach. The behaviour of a river can, therefore, result in the variation of the shape of the river cross-section and/or its plan-form. Aggradation, degradation, scour and deposition of sediment around bends, and meandering are a few examples of such changes.

Bends

With slight asymmetry in flow, an alluvial river tends to develop bends which are characterised by scour and erosion of sediment on the concave (*i.e.,* outer) bank and deposition of sediment on the convex (*i.e.*, inner) bank. Because of curved flow lines around the bend, the flow is subjected to centrifugal forces and, hence, there is a transverse slope of the water surface due to the superelevation of the water surface at the concave bank. As a result, the bottom water (moving with relatively smaller velocity) moves from the concave bank to the convex bank and also carries with it the bed material and deposits it near the convex bank. To replace this bottom water, water dives in from the top at the concave bank and flows along the bottom carrying sand and silt to the convex bank where it is deposited. This secondary motion is primarily responsible for the erosion of the sediment on the concave bank and the deposition of the sediment on the convex bank. The depth of flow in a river at the bend thus becomes deeper at the concave bank (Fig. 12.4).

Meanders

The continued action of the secondary flow developed around river bends causes further erosion and deposition of the sediment, respectively, on the concave and convex banks of the river. Thus, the river bends become more sharp and the river attains a meander pattern and becomes a meandering river. Meander patterns are usually associated with wide flood plains comprising easily erodible material.

There have been several attempts to explain the mechanism of meander development. According to Inglis (2), "Meandering is nature's way of damping out excess energy during a wide range of varying flow conditions, the pattern depending on the grade of material, the relation between discharge and charge (sediment load), and the rates of change of discharge and charge". Thus, a channel having excess energy attempts to increase its length by meanderingthereby decreasing its slope.

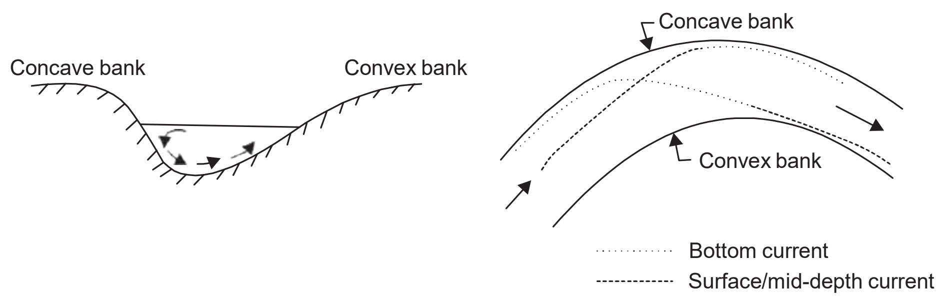


Fig. 12.4 Movement of water at a bend

Joglekar (3) and other Indian engineers do not agree with the theory of excess energy. According to them (3), the primary cause of meandering is excess of total sediment load during floods. A river tends to build a steeper slope by depositing the sediment on the bed when the sediment load is in excess of that required for equilibrium. This increase in slope reduces the depth and increases the width of the river channel if the banks do not resist erosion. Only a slight deviation from uniform axial flow is then required to cause more flow towards one bank than the other. Additional flow is immediately attracted towards the former bank, leading to shoaling along the latter, accentuating the curvature of flow and finally producing meanders in its wake.

Meanders can be classified (4) as regular and irregular or, alternatively, as simple and compound. Regular meanders are a series of bends of approximately the same curvature and frequency. Irregular meanders are deformed in shape and may vary in amplitude and frequency. Simple meanders have bends with a single radius of curvature. In compound meanders each bend is made up of segments of different radii and varying angles.

The geometry of meanders can be described by the meander length M_L and the width of the meander belt M_B (Fig. 12.2), or by the sinuosity or the tortuosity. Many investigators have attempted to relate the geometry of meanders with the dominant discharge. The *dominant discharge* is defined (5) as that hypothetical steady discharge which would produce the same result (in terms of average channel dimensions) as the actual varying discharge. Inglis (2) found that for north Indian rivers, the dominant discharge was approximately the same as the bankful discharge and recommended that the dominant discharge be taken as equal to half to two-thirds of the maximum discharge.

Inglis (2) gave the following relationships for M_L and M_B (both in metres) in terms of the dominant discharge (or the bankful discharge) Q (in m^3/s) for rivers in flood plains:

$$M_L = 53.6\, Q^{1/2} = 6.06\, W_s \tag{12.1}$$

$$M_B = 153.4\, Q^{1/2} = 17.38\, W_s \tag{12.2}$$

Here, W_s (in metres) is the bankful width of river (Fig. 12.1).

Agarwal *et al.* (6) have re-examined the laboratory and field data for discharges ranging from 9×10^{-6} to 10^4 m^3/s and found that the following relationships proposed by them are better than Inglis' relationships:

$$M_L = 29.70\, Q^{0.32} \quad \text{for } Q < 9 \text{ m}^3/\text{s} \tag{12.3}$$

$$M_L = 11.55\, Q^{0.75} \quad \text{for } Q > 9 \text{ m}^3/\text{s} \tag{12.4}$$

and

$$M_B = 0.476\, M_L \tag{12.5}$$

The sinuosity of a river is defined as the ratio of *talweg* length to the valley length. Joglekar (3) defines tortuosity as,

$$\text{Tortuosity} = \frac{\text{talweg length} - \text{valley length}}{\text{valley length}} \times 100$$

The sinuosity varies from 1.02 to 1.45 over 1500 km length of the river Indus. For the river Ganga, the sinuosity varies from 1.08 to 1.51 (3). Table 12.1 shows the extent of tortuosity of the river Ganga.

Table 12.1 Tortuosity of the river Ganga in different reaches (3)

Reah	*Valley length (km)*	*Talweg length (km)*	*Tortuosity (%)*
Balawalli to Garhmukteshwar	104.6	117.5	12
Garhmukteshwar to Rajghat	59.5	67.6	14
Rajghat to Kanpur	281.6	313.8	11
Kanpur to Allahabad	117.0	217.3	23
Allahabad to Varanasi	137.8	209.2	51
Varanasi to Sara	684.0	869.0	27
Sara to the Bay of Bengal	297.7	321.9	8

It should be noted that the meandering pattern of alluvial rivers is commonly encountered in alluvial stream. River training methods are generally adopted for meandering rivers. It may also be noted that the meander pattern is not stationary and moves slowly in the downstream direction.

Cutoffs

Cutoffs can be defined (3) as a process by which an alluvial river flowing along curves or bends abandons a particular bend and establishes its main flow along a comparatively straighter and shorter channel. During the development of meanders, there is always a lateral movement of the meanders due to their gradual lengthening. Increased frictional losses and bank resistance tend to stop this lateral movement. When the bend and the bank resistance become too large for continued stretching of the loop, the flow finds it easier to cut across the neck than to flow along the loop (Fig. 12.5). This results in a cutoff. Cutoff is, thus, a natural way of counter-balancing the effect of the ever-increasing length of a river course due to the development of meanders. Usually, a river has shallow side channels within the neck of the meander loop. These side channels may either be part of the main channel of an earlier river course or are formed by floods spilling over the banks of the river channel. Cutoffs can develop along these shallow side channels. Alternatively, cutoff may be artificially induced for some other purpose.

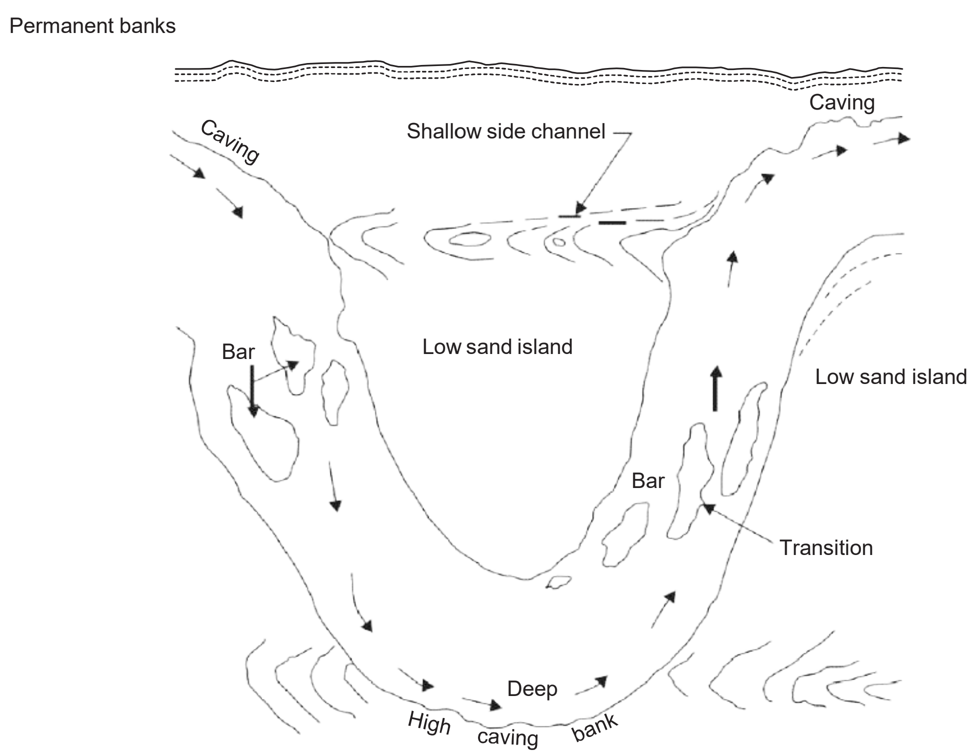


Fig. 12.5 A river bend

The rapidity with which a cutoff channel develops depends on local conditions. For example, the cutoff on Chenab near Shershaha took one year but the great Golbethan bend of the Ganga downstream of Hardinge Bridge began to cutoff in 1911, and the cutoff developed only after five years (7).

Whenever a river succeeds in establishing a cutoff, there follows a period of non-equilibrium for long distances upstream and downstream of the newly-formed channel. Banks start caving in and new channels are formed while some other channels get silted up. Only after a couple of floods, the equilibrium is, once again, established.

Sometimes it is advantageous to make a controlled artificial cutoff to avoid the chaotic or non-equilibrium conditions when a natural cutoff develops. An artificial cutoff reduces flood levels and flood periods. Artificial cutoffs have been used to shorten the travel distance and increased ease of manoeuvring of boats along the bend during navigation. In such situations, use of training measures like groynes and revetment on banks usually becomes necessary to prevent bank erosion and arrest the natural tendency of the river to meander.

For inducting an artificial cutoff, a suitable pilot cut (or pilot channel) of small cross-section is initially made so as to carry 8 to 10 per cent of the flood discharge (5). The pilot channel is then allowed to develop by itself and sometimes such gradual development is assisted by dredging. Pickles (8) has made the following recommendations for design and execution of artificial cutoffs:

(i) The pilot channel should be tangential to the main direction of river flow approaching and leaving the cutoff.

(ii) The pilot channel is usually made on a mild curve, the curvature being less than the dominant curvature of the river itself.

(iii) Entrance to the pilot channel is made bell-mouthed. Such transition at the exit is considered unnecessary because the cut develops first at the lower end and works progressively upstream.

(iv) The cut, when unlikely to develop because of either coarseness of the material or low shear stress, should be excavated to average river cross-section.

(v) The width of the pilot cut is unimportant as the cut ultimately widens due to scouring. Hence, in practice, the width is determined by consideration of the type and size of the dredging equipment used.

(vi) When a series of cutoffs is to be made, the work should progress from the down-stream to upstream.

Lateral Migration of Alluvial Rivers

Some alluvial rivers have shown a tendency for lateral migration over a period of years. Some rivers, which might have had very little lateral movement over a long period of time, may suddenly start migrating laterally during a succeeding period. Such changes in river courses have been found in the Yellow river, and in other rivers of China (5). Lateral movement of the Kosi river in India offers a classic example of the lateral migration of alluvial rivers (3). The Kosi river brings in a large amount of sediment load which it is unable to transport. In the process of building up an inland delta in the plains, it shifted over 110 km westward during the period from 1736 to 1964 (3). However, it did not shift from its original position at the Belka hills (near Chatra where the river leaves the Himalayas and enters the Gangetic plains) and also at Kursela (close to the confluence of the Kosi and the Ganga rivers). The lateral migration of the Kosi river was arrested by the construction of levees. Table 12.2 gives the average rates of lateral migration of the Kosi river for the period 1736-1950.

Observations of the migration of the Kosi river and the Yellow river indicate that the lateral migration of alluvial rivers is mainly due to the excessive sediment load during the floods, large variation in the river discharge, large slope, and geologically young rock formations (5).

Table 12.2 Average rates of migration of the Kosi river (9)

Period	*Period of Movement (years)*	*Approximate distance moved (km)*	*Rate of movement (km/year)*
1736-1770	34	10.8	0.32
1770-1823	53	9.3	0.18
1823-1856	33	6.1	0.18
1856-1883	27	12.9	0.48
1883-1907	24	18.5	0.77
1907-1922	15	10.9	0.73
1922-1933	11	29.0	2.63
1933-1950	17	17.7	1.04

Delta Formation

As a river approaches the sea, it dumps its sediment load into it and the river mouth extends towards the sea on account of the sediment deposition. This lengthening of the river further reduces the already small slope of the river at its mouth. Thus, the sediment transporting capacity of the river decreases and the river deposits the sediment on the bed and banks of the river channel raising the river stage. In general, the rise in the river stage results in spilling over the banks and cutting through the banks if they are not sufficiently resistant. The spilled water may form branch channels or spill channels which, after their full development, start behaving like the parent river. As a result, the sediment transport capacity decreases considerably and the river bed starts rising resulting in the formation of delta (Fig. 12.6).

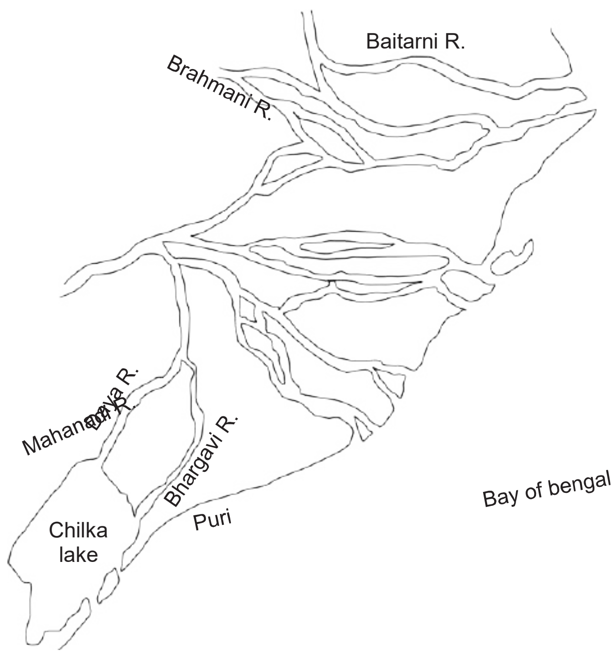


Fig. 12.6 Delta formation of the Mahanadi river

When the sediment load in a river is in excess of its sediment-transporting capacity, the excess sediment gets deposited on the river bed. Therefore, the sediment load entering a given reach is greater than the sediment load leaving the reach during the same time. This causes a rise in the bed level (and, hence, the flood level) and increase in the bed slope with time. The phenomenon itself is known as aggradation. Aggradation usually occurs because of an increase in the sediment load at a section without change in discharge and sediment size. Aggradation occurs most commonly upstream of a reservoir. Because of the construction of a dam, the sediment-transporting capacity of the river upstream of the reservoir is reduced. The coarser sediment deposits farther away from the dam while the finer sediment is deposited closer to the dam. Aggradation can also occur when a tributary brings into the main river sediment load in excess of the river's sediment-carrying capacity. Also, when rivers are divided into two or more branches, as in the case of deltaic rivers, it may not be possible for every branch to maintain the state of equilibrium in respect of sediment flow, and, hence, aggradation may result. If an offtaking canal takes relatively sediment-free water, it would result in aggradation in the main river downstream of the offtake point.

Degradation

When the amount of sediment load being transported by a river is less than the sediment-transporting capacity of the river, the excess sediment needed to satisfy the capacity of the river will be eroded from the erodible bed, thereby lowering the bed. If the banks are also erodible, widening of the river would also result. This phenomenon of erosion of bed and banks is known as degradation. Degradation occurs in streams downstream of a reservoir. The reservoir stores a large percentage of the sediment load carried by the river. The flow downstream of the reservoir is relatively free of sediment, and, therefore, cause degradation resulting in lowering of the bed levels.

An interesting example of large bed level changes is that of degradation on the Ratmau torrent downstream of a level crossing on the Upper Ganga canal in UP (10). The flood discharge in the Ratmau torrent has been of the order of 1000 m^3/s with a maximum of 2250 m^3/s in 1947. The level crossing was constructed in 1850 to pass the torrent across the canal carrying a discharge of 300 m^3/s. During the period from 1854 to 1977, the bed level of the torrent immediately downstream of the level crossing was lowered by 8.0 m. The bed slope of the torrent downstream of the crossing has decreased from 1.56×10^{-3} to 5.5×10^{-4}. The lowering of the bed required frequent modifications in the stilling basin (10).

If degradation has occurred in a river near its confluence with a tributary, the tributary slope increases. Thus the sediment-transporting capacity of the tributary increases near the confluence, resulting in degradation near the confluence and this effect extends further upstream and also to other sub-tributaries. Degradation can also occur if the water discharge in a river is increased without increase in sediment load or sediment size. If the sediment-free waste water of an irrigation project is added to a river, it may result in degradation in the river.

Downstream of a spillway, a hydraulic jump is usually formed to dissipate the excess energy. Due to lowering of the bed and water levels on account of degradation, jump may shift downstream of the apron and endanger its safety. Dams on pervious foundations are subjected to uplift pressure which depends on the effective head which is equal to the difference between the reservoir and tail-water levels. The lowering of the tail-water level due to degradation increases the effective head and, hence, the uplift pressure on the dam. Lowering of water level at the intakes on account of degradation may make the diversion of water for irrigation more difficult. Lowering of bed in navigable rivers due to degradation may, at times, make the navigation locks inoperative.

Degradation can sometimes be beneficial too. Because of lowering of tail-water levels, the effective head increases. This would cause an increase in hydro-electric power generation. Degradation increases the river capacity to carry the flood flow. This lowers the high flood levels of a river. Lowering of the water level in a river on account of degradation lowers the ground water table in the adjoining areas.

RIVER TRAINING

River training includes all such measures as are taken for controlling and regulating river flow and river configuration. River training works are constructed either across a river, or along it. River training structures include levees or embankments built along the river to contain floods, and spurs and guide banks are constructed for altering the local flow conditions and guiding the flow. Besides, a river can be dredged to train it for navigation purposes. A river can also be trained by diverting its flow into a secondary channel or by executing artificial cutoffs on the main river so as to cause reduction in flood levels. Bank protection measures are also included in river training methods.

12.4.1. Objectives of River Training

River training measures aim at achieving one or more of the following objectives:

(i) Flood Protection

River floods of very small frequency inundate the fertile and thickly-populated plains adjacent to the river, and, thus, cause considerable loss to human life, property, agriculture, and public and private utilities.

During the years of large floods, damage is likely to be several times more. Flood control measures for thickly-populated flood plains, therefore, become essential, even if these measures do not assure complete protection under all conditions. River training for flood protection, also known as 'high water training' or 'training for discharge,' is achieved by one or more of the following four methods :

(a) Construction of levees or embankments to confine water in a narrower channel,

(b) Increasing the discharge capacity of natural channels by some means such as straightening, widening or deepening,

(c) Provision of escapes or diversion from the main channel into an auxiliary channel for water in excess of the carrying capacity of the main channel, and

(d) Construction of reservoirs.

(ii) Navigation

For a river to be navigable, sufficient depth and width required for navigation should be available even at low water level in the river. River training for navigation is also known as 'low water training' or 'training for depth'. Measures to achieve adequate depth in a river for navigation include dredging the shallow reaches of the river and using spurs to contract the river channel, thus, increasing its depth. Sometimes, low flow is supplemented from another source to achieve the desired depth and width. Canalisation makes a non-navigable river navigable, and, is accomplished by building a series of small dams or weirs and locks. Sharp curves along the river need to be eliminated so that ships can move easily.

(iii) Sediment Control

River training for sediment control is also called 'mean water training' or 'training for sediment'. This type of training aims at rectification of river bed configuration and efficient movement of sediment load for keeping the channel in a state of equilibrium (3). River training methods for this purpose involve construction of such structures which would induce the desired local curvature to the flow. Spurs and pitched islands are normally used for training the river for sediment.

(iv) Guiding the Flow

Hydraulic structures, such as canal headworks, and communication structures such as bridges, have to be protected against outflanking and the direct attack of flow. This requires training of the river over its considerable reach by building a system of guide banks, known as Bell's guide banks, on one or both sides of the stream at the bridge site. The purpose of these guide banks is to make sure that water flows between the abutments of the bridge. The spacing between these guide banks conforms to the width required for the river to pass the design flood discharge. Similarly, guide banks are provided to guide the flow at the weir site. Marginal bund and lateral spurs guide the flow through the guide banks.

Sometimes the flow in a river needs to be deflected away from a bank in order to protect some portions of the river bank or for contracting the river. This is done by constructing one or more spurs projecting into the river from its banks.

(v) Stabilization of River Channel

Weak river banks, which are likely to cave in or get eroded, need to be protected by training methods, such as stone pitching, lining, and so on. In some cases, the stability of the bed may also be endangered in some reaches due to increase in the bed shear on account of local flow conditions.

RIVER TRAINING METHODS

The planning and design of river training structures is accomplished by using empirical methods and reliance has to be placed on the intuition and judgement of experienced engineers. Model investigations are also resorted to for finalising the plans and design of river training structures. Commonly used methods of river training have been briefly described in the following sections.

Levees

A levee (also known as an embankment, bund, marginal bunds or dike) is an embankment running parallel (or nearly so) to the river and is constructed to protect the area on one side of it from flooding. The method of constructing levees on one or both sides of a river to contain the flood within the leveed portion is the oldest and most commonly used method of flood control. Levees along the Nile river in Egypt were constructed prior to 600 BC. Levees have been constructed recently on many important rivers of the world such as the Ganga, the Kosi, the Mahanadi and the Gandak in India, the Yellow, the Pearl, the Yangtze and the Huai in China, the Mississippi in the USA, and the Danube and the Rhine in Europe.

The alignment of levees for a river is decided by the location of important cities, industries, and other areas along the river which need to be protected against floods. Closely-spaced levees will be very high and, hence, massive and uneconomic. Hence, levee spacing is also governed by economic considerations. Levees should be located farther apart considering: (*i*) the desirability of having high discharge capacity of river for a given stage, and (*ii*) the requirement that the entire meander belt be within the levees so that they are not strongly attacked by the river. The levees should, obviously, have the general curvature of the river so that the river does not attack the levees.

The design of a levee is similar to that of an earth dam. It should, however, be noted that while the upstream face of an earth dam is exposed to water most of the time, that of a levee is exposed to water for a very short period during the flood season only. The top width of a levee is generally kept between 3 to 8 m or more depending upon the levee height. The levee height is decided such that the levee is able to contain a flood of a reasonable return period of, say about 500 years (5). The flood stage at any section of a river corresponding to such a flood can be obtained by routing the flood through the river. A freeboard of 1-2 m is added to the flood stage to obtain the elevation of the top of the levee. The probable settlement of levee after its construction should also be accounted for while determining the levee height. The side slopes of levees vary from $1V : 2H$ to $1V : 6H$. In case of high levees, berms are also provided on the land-side slope.

One of the major effects on regime of river due to levee construction is the reduction in the river width and, hence, increase in velocity of flow. As such, the sediment, which would have deposited on the river bed/flood plains in the absence of levees, is now carried downstream and deposited either in an unleveed portion or in the sea. Other effects of confining the flood within levees are as follows (12):

(i) Increase in the rate of travel of flood wave in the downstream direction,

(ii) Rise in the water surface elevation in the river during flood,

(iii) Reduction of storage and, hence, an increase in the maximum discharge downstream, and

(iv) Decrease in the water surface slope of the stream above the leveed portion as a result of which aggradation occurs upstream of the leveed reach.

Failure of levees can be due to one or more of the following causes (12):

(i) Overtopping,

(ii) Erosion of riverside slope by river current,

(iii) Caving in of the banks,

(iv) Infiltration through the foundation,

(v) Infiltration through the embankment,

(vi) Leaks as a result of holes dug by rats, crabs, and white ants, or from rotten roots and cracks due to shrinkage of soil,

(vii) Loosening of the embankment by wind action on large trees planted on it, and

(viii) Human action.

As such, levees are very susceptible to failure during floods. Hence, continuous supervision, particularly during floods, and availability of enough labour and material on the spot are necessary to detect and plug breaches in the levees, if they occur.

The method of flood control by levees is fairly simple and economical as it uses locally available material and labour for its construction. Besides, levees can also be extended gradually to cover more and more area.

Spurs

Spurs (also known as groynes, spur dikes, or transverse dikes) are structures constructed in a river transverse to the river flow, extending from the bank into the river (3). Spurs guide the river flow, promote scour and deposition of the sediment where desired, and trap the sediment load to build up new river banks. Spurs are generally made from locally available earth. The nose (or head) and the sloping faces of the spurs must be protected against wave action by hand-placed rubble facing. Stone apron is provided to prevent the failure of spurs due to excessive scour at the nose and sides. Spurs are probably the most widely used river training structures and serve the following function in river regulation:

(i) Training a river along the desired course by attracting, deflecting or repelling the flow in the river channel,

(ii) Creating a slack flow with the object of silting up the area in the vicinity of spur,

(iii) Protecting the river bank by keeping the flow away from it, and

(iv) Contracting a wide river channel for the improvement of depth for navigation.

Spurs can be used either singly or in series or in combination with other river training measures. The design of spur depends on the following:

(i) River discharge,

(ii) Angle of attack,

(iii) Sediment load,

(iv) Meander length,

(v) Curvature of the river, and

(vi) Upstream and downstream river training measures.

Spur length is usually restricted to less than 20% of the river width to avoid adverse effects on the opposite bank and, at the same time, the spur length is kept longer than 1.5 to 2 times the depth of flow (13). Shorter spur length in deeper rivers induces swirling motion on both the upstream and downstream sides of the spur. This swirling motion may extend up to the adjacent river bank and cause the bank erosion necessitating bank protection measures. The spacing of spurs in a wide river is larger than that in a narrower river for similar conditions. A larger spacing can be satisfactory for convex banks and a smaller spacing is desirable at concave banks. At crossings (*i.e.,* the straight reach between two consecutive bends of a river), an intermediate spacing can be adopted. Spacing between adjacent spurs is generally kept between 2 and 2.5 times the spur length (14). Ahmad (15) has suggested that spurs used for bank protection be spaced at five times their length. However, spurs used in navigation channels are generally spaced at 0.75 to 2 times their length (16). Maintenance of the nose of longer spurs during floods would generally be difficult as has been experienced in the past on the rivers Kosi and Gandak. Moreover, a longer spur would result in relatively higher afflux on the upstream side of the spur and may induce excessive seepage through the spur which may lead to piping and breach in the spur. Such breaches have indeed occurred in the rivers Kosi and Gandak (17). The top width of a spur would be between 3 and 6 m and a freeboard of 1 to m above HFL should always be provided in case of non-submerged spurs (14). Slopes on the upstream shank and nose should be $1V : 2H$ and the slope on the downstream face may be $1V : 1.5H$ to $1V : 2H$ (14) (Fig. 12.7). Stone pitching on the slopes of a spur is placed manually as per the standard practice. A graded filter 20 to 30 cm in thickness, satisfying the standard filter criteria should be provided below the pitching (14). A launching apron (Art. 12.5.3.1) should also be provided to protect the stone pitching.

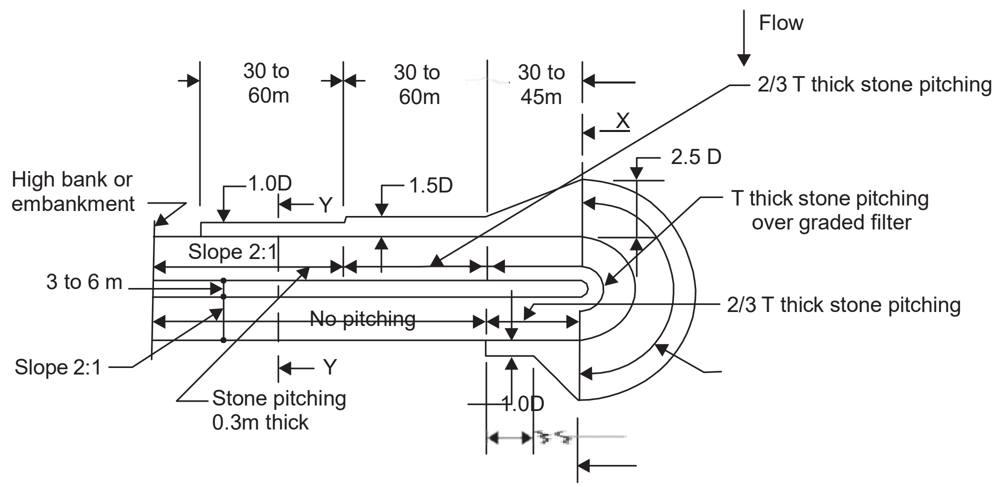
It is always advisable to finalise the spur designs only after conducting model studies. Spurs can be classified as follows:

(i) Classification based on the methods and material of construction : permeable and impermeable.

(ii) Classification based on the height of the spur with respect to high flood level : Submerged and non-submerged.

(iii) Classification based on the functions : attracting, deflecting, repelling and sedimenting, and

(iv) Special types : Denehy's T-headed groynes, hockey type, *etc.*



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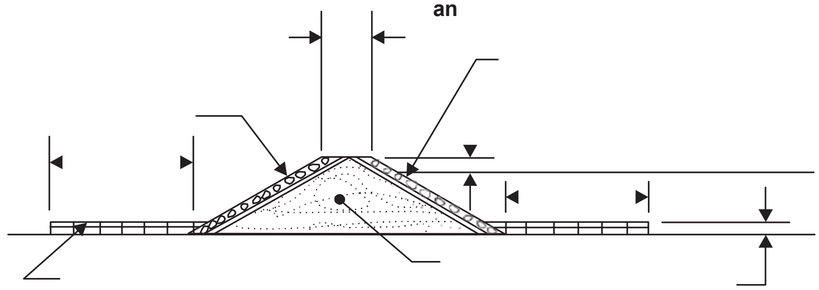


3 to 6 m

Graded filter 20 to 30 cm

T thick stone
pitching

1.0 To 1.5 m Free board

~~1.5 D max~~

HFL

2:1

2:1

~~1.5 D max~~

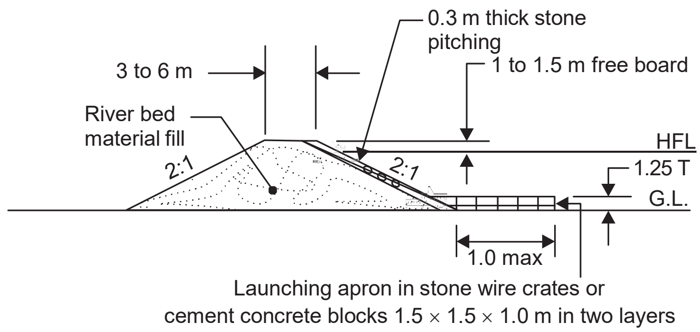
LWL

River bed material fill

Launching apron in stone wire crates
or cement concrete blocks 1.5 × 1.5 × 1.0 to 1.5 m in two layers

1.25 To 1.5 T

Enlarged section XX



Enlarged section YY

Note : See Fig. 12.10 and Eq. (12.7) for meanings of D and T.

Fig. 12.7 A typical impermeable spur

A series of permeable spurs reduces the flow velocity between the spurs which results in deposition of the sediment carried by the river water. Such spurs are, therefore, more suitable for rivers carrying heavy sediment load. In rivers carrying clear water, these spurs dampen the erosive strength of the current and thus prevent local bank erosion. Experience has indicated that permeable spurs are more effective than solid spurs for regulating the river course or protecting the banks and levees, especially in a sediment-laden river flow (3). Further, flow through the permeable spurs does not change abruptly (as it does in passing around a solid spur) and, hence, does not cause serious eddies and scour holes. Permeable spurs can be either submerged or non-submerged. These spurs are relatively cheap, but, are not strong enough to resist shocks and pressures from debris, floating ice, and logs and are, therefore, unsuitable for the upper reaches of a river (3).

Usually, permeable spurs are either tree spurs or pile spurs. A tree spur has a thick wire rope of about 25 mm diameter. This wire is firmly anchored to the bank at one of its ends and tied to a heavy buoy or concrete block at the other end (Fig. 12.8). Leafy trees with strong stem and branches are tied to the main rope by subsidiary ropes through holes drilled in the tree stem. The trees should be packed as closely as possible. When trees become heavy due to entrapped sediment, they sink.

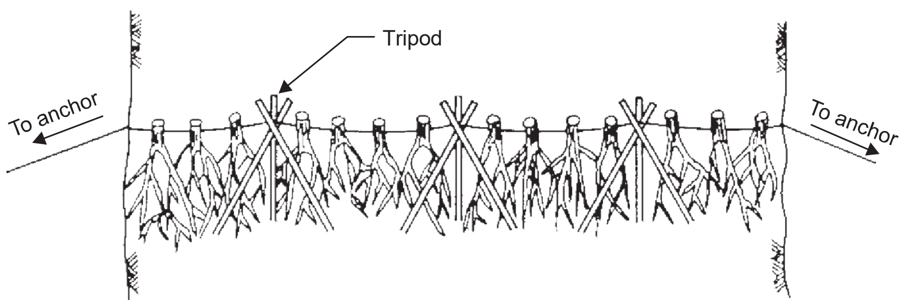


Fig. 12.8 A typical tree groyne

Pile spurs are constructed (3) by driving piles of timber or RCC or sheet piles up to about 6 to 9 m inside the river bed. These piles are 2.4 to 3 m apart and form at least two to three rows. Each row of piles is closely inter-twined either by brushwood branches or by horizontal railings. The upstream row is braced to the downstream row by transverses and diagonals. The space between the rows of piles is filled by alternate layers of 1.8 m thick brushwood weighted by 0.6 m thick boulders or sand bags. The filling should not be completely of stone since the spur is intended to be permeable to start with. Deep scour holes developed at the nose of these spurs do not cause danger because stones from the face of the spur fall into the scour hole and create a blanket which prevents undermining.

Impermeable or solid spurs are constructed as either rockfill or earth-core embankment armoured with a scour resistant surface. These can be made to attract or repel the flow away from the bank along the desired course. The side slopes vary from $1V{:}1H$ to $1V{:}5H$ depending on the material of construction (5). The nose of the spur is usually flat with a slope of $1V{:}5H$. Spurs used in river training for navigation are generally kept straight. But, other shapes of spurs, such as hockey spurs and T-shaped, have also been used. Shapes of scour holes for different types of spurs have been shown in Fig. 12.9. Obviously, T-head spur (first constructed by Denehy at the Okhla headworks on the Yamuna river and, hence, also known as Denehy's spur) requires stone apron protection against scour for relatively small area and, therefore, is most economical. T-head spurs have also been effectively used on the Ganga river at the Narora headworks.

Spurs meant for contracting the river channel are generally oriented with their axes normal to the current. Sometimes, spurs are oriented to point upstream, the advantage of which is that during the flood the flow is directed towards the centre of the river. Thus, the strong currents are kept away from the flood plains and flood dikes. Such spurs, pointing upstream, are also called repelling spurs. On the other hand, spurs pointing in the downstream direction attract the flow towards the bank and are, therefore, known as attracting spurs. A repelling spur produces a more desirable curvature to the flow downstream, leading to pronounced deposition (5). Besides, such a spur has large stagnation region on the upstream side and is, therefore, able to protect a greater length of the bank than that protected by the attracting spur. The repelling spur is usually inclined at 5° to 20° (to the line normal to the bank) in the upstream direction.

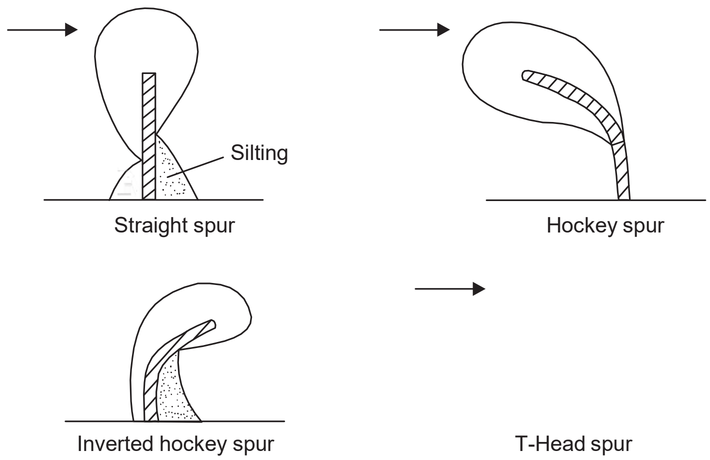


Fig. 12.9 Scour patterns for different spurs

Guide Bank

While selecting a site for bridge on an alluvial river, certain site 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).

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 (3). 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 (18) and Gales (19).

The first step in the design of a bridge 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 Eq. (8.29). The overall waterway between the guide banks is obtained by adding the thickness of piers to the clear waterway. Sharma and Asthana (20), based on their studies of design and performance of 20 bridges constructed during 1872-1966 recommended waterway width (in metres) varying from 3.3 $\sqrt{Q}$ to 7.1 $\sqrt{Q}$ in the alluvial stage and from 2.4 $\sqrt{Q}$ to 4.8 $\sqrt{Q}$ in the boulder stage. Here, Q is the river discharge in m^3/s. Obviously, a smaller waterway would cause a large afflux resulting in danger of outflanking.

Figure 12.10 shows the plan and sections of a typical guide bank. In plan, the guide bank can be either parallel, converging upstream or diverging upstream. In general, guide banks diverging upstream need to be longer than the straight or converging guide banks (5). Flows with acute curvature result in shoal formation (21) near the shanks [Fig. 12.11 (*a*)]. Hence, one may consider providing elliptical shanks, [Fig. 12.11 (*b*)] instead of straight shanks. However, there is a possibility of shoaling, away from the elliptical guide bank, due to the divergence and this should be studied carefully in a model before finalising the design.

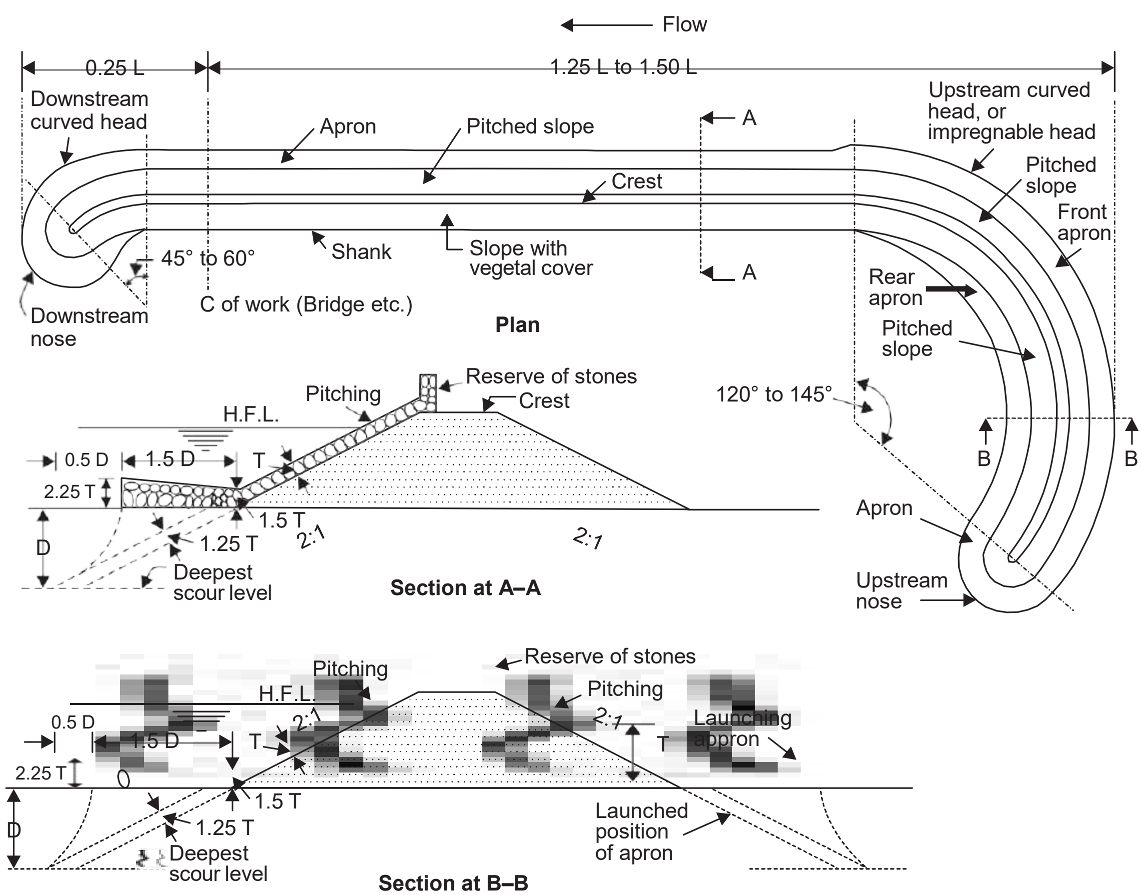


Fig. 12.10 Guide bank

The length of guide banks upstream of the bridge should be 1.1 times the length of the bridge (18). Gales (19), however, has recommended that this length should be between 1.25 and 1.5 times the bridge length for flood discharges ranging from 7000 to 70,000 m^3/s. The length of guide banks downstream of the bridge should be about 0.25 times the bridge length (5).

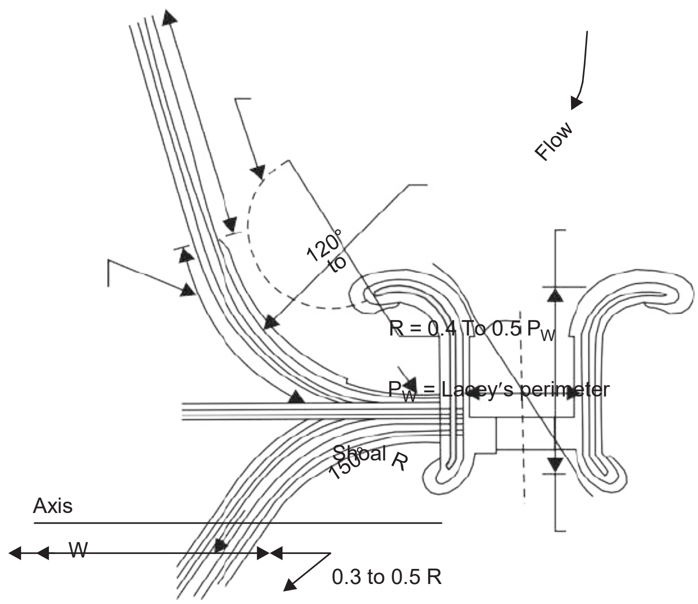


Fig. 12.11 (*a*) Flow near a straight guide bund with circular head

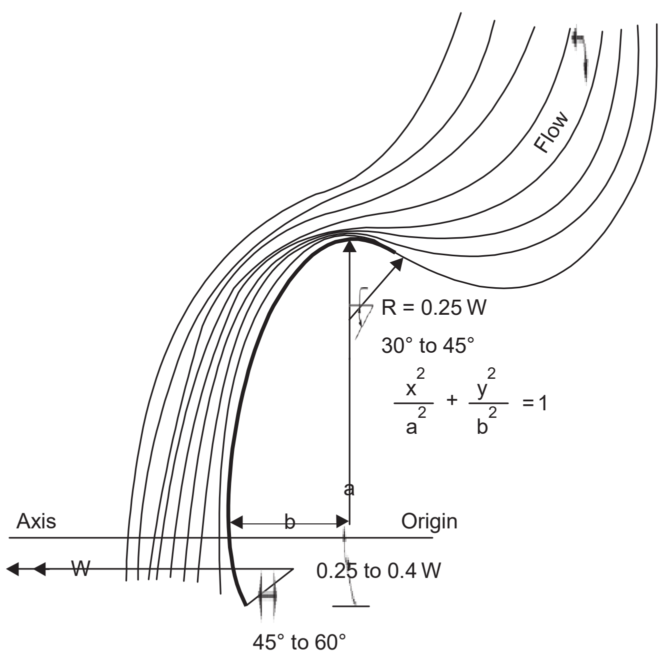


Fig. 12.11 (*b*) Flow near an elliptical guide bund

Radius of the upstream curved head R_1 should be equal to $2.2\sqrt{Q}$ metres. A smaller radius is permissible for smaller rivers. The sweep angle generally varies between 120° and 145° for the upstream curved head. The radius of the downstream curved head R_2 is generally kept equal to $1.1\sqrt{Q}$ metres. A sweep angle of 45° to 60° for the downstream curved head is considered satisfactory.

The elevation of the guide banks is obtained by adding a freeboard of 1.5 to 2.5 m to the high flood level of 100-year-flood. Alternatively, a freeboard of 1.0 m is added to the high flood level of a 500-year flood to determine the elevation of the top of guide bank. The top width of guide banks is generally fixed between 6 to 9 m (5). Both faces of the guide banks have side slopes of $1V{:}2H$ or flatter. Locally available sand, silt, and gravel are used for the construction of the core of the guide banks. To protect the face towards the river and the back of the curved heads of the guide banks from severe erosion, large stone pitching is provided. The embankment - side faces of the guide banks, however, do not need such protection. The stones used for slope protection must be large enough to withstand the force of current and stay in place. The minimum size of stones of relative density 2.65 required for this purpose can be calculated by the empirical relation (2, 22),

$$d = 0.023 \text{ to } 0.046U^2 \tag{12.6}$$

where, d is in metres and U is in m/s. Normally, angular and graded stones having the ability to interlock and weighing between 450 and 1800 N are used for slope protection. The thickness of stone pitching T in metres is related to river discharge Q (in m^3/s) by an empirical equation as follows (2, 5, 22):

$$T = 0.04 \text{ to } 0.06Q^{1/3} \tag{12.7}$$

For large streams, the constant 0.06 should be reduced to 0.04 (5). Thickness for pitching should be increased by 25 per cent for the curved head region. The pitching must be provided on both sides on the guide bank embankments in the curved head regions.

12.5.3.1. Launching Apron

Heavy scour of the river bed at the curved heads and shanks of guide banks can cause undermining of the stone pitching thereby resulting in failure of the guide banks. Such failure of guide banks can be prevented by providing launching aprons (consisting of stones) beyond the toe of the guide banks as shown in Fig. 12.10. As the scour continues, the launching apron is undermined and it eventually covers the face of the scour hole adjacent to the guide bank. The slope of the apron in the launched position varies between $1(V) : 1.25(H)$ and $1(V) : 2.5(H)$. For the purpose of design, a slope of $1(V) : 2(H)$ for loose boulders and $1(V) : 1.5(H)$ for concrete blocks can be assumed (23). The scour depths (below HFL) in the vicinity of the guide banks can be taken as K times Lacey's normal scour depth given by Eq. (8.32) or Eq. (8.33). The value of K is taken as 2.25 to 2.75 at the upstream nose and 1.5 to 2.0 at the downstream nose of guide banks, 1.25 for transition from the straight portion to the nose of guide bank and 1.5 to 2.0 for straight reach of guide banks (2, 5, 22). The width of the launching apron is generally kept equal to 1.5 times the scour depth (below the bed) at that place. The stone requirement for the launching apron is computed on the assumption of uniform apron thickness of 1.25 T in its launched position. Thus, if D is the depth of scour below the bed, the quantity of stone required for launching apron of 1 m length (along the guide bank) would be $\sqrt{5}\,D \times 1.25T$ *i.e.,* 2.8 DT m^3. This volume is provided in the form of a wedge (to account for the non-uniformity of stone layer thickness in launched condition) as shown in Fig. 12.10. The launching apron should be provided on both sides of the guide bank embankments in the curved head regions. No spur should project from a guide bank. For maintenance purposes, a reserve of stones is usually kept ready on the top of the guide bank for dumping, if the bank is threatened.

Generally, a filter is always provided below the stone pitching provided for the bank protection. However, as per the earlier practices, the filter was generally, not provided between the launching apron and the river bed. It was believed that filter might hinder the launching of the apron. In the absence of a filter between the launching apron and the river bed, finer particles from the bed near the toe of the bank would escape through the voids in the apron which may result in vertical sinking of the apron and the toe of the bank gets exposed for possible failure of the bank protection. Hydraulic model studies (24) have indicated that the provision of filter below launching apron does not hinder its launching. During the launching of the apron, however, some filter material may get carried away downstream by the flow. Therefore, one should provide a relatively thicker filter layer. Or, alternatively, one may provide synthetic fibre filter.

Example 12.1 Design guide bunds (or banks) and launching apron required to be provided for a bridge across a river whose total waterway is 658.88 m. The design flood discharge is 13100 m^3/s which may be increased by 20% for the design of launching apron. Mean size of the river bed material is 0.3 mm.

Solution: The geometric parameters of the guide bunds are calculated from the equations given by Spring (18).

Length of the guide bund upstream of the bridge, L_1 is given as

$$L_1 = 1.1\ L = 1.1 \times 658.88 \cong 725.0 \text{ m}$$

Length of the guide bund downstream of the bridge, L_2 is given as

$$L_2 = 0.25\ L = 0.25 \times 658.88 \cong 165.0 \text{ m}$$

The radius of the upstream curved head, R_1 is given as

$$R_1 = 2.2\sqrt{Q} = 2.2\sqrt{13100} = 251.8 \text{ m} \cong 250 \text{ m}$$

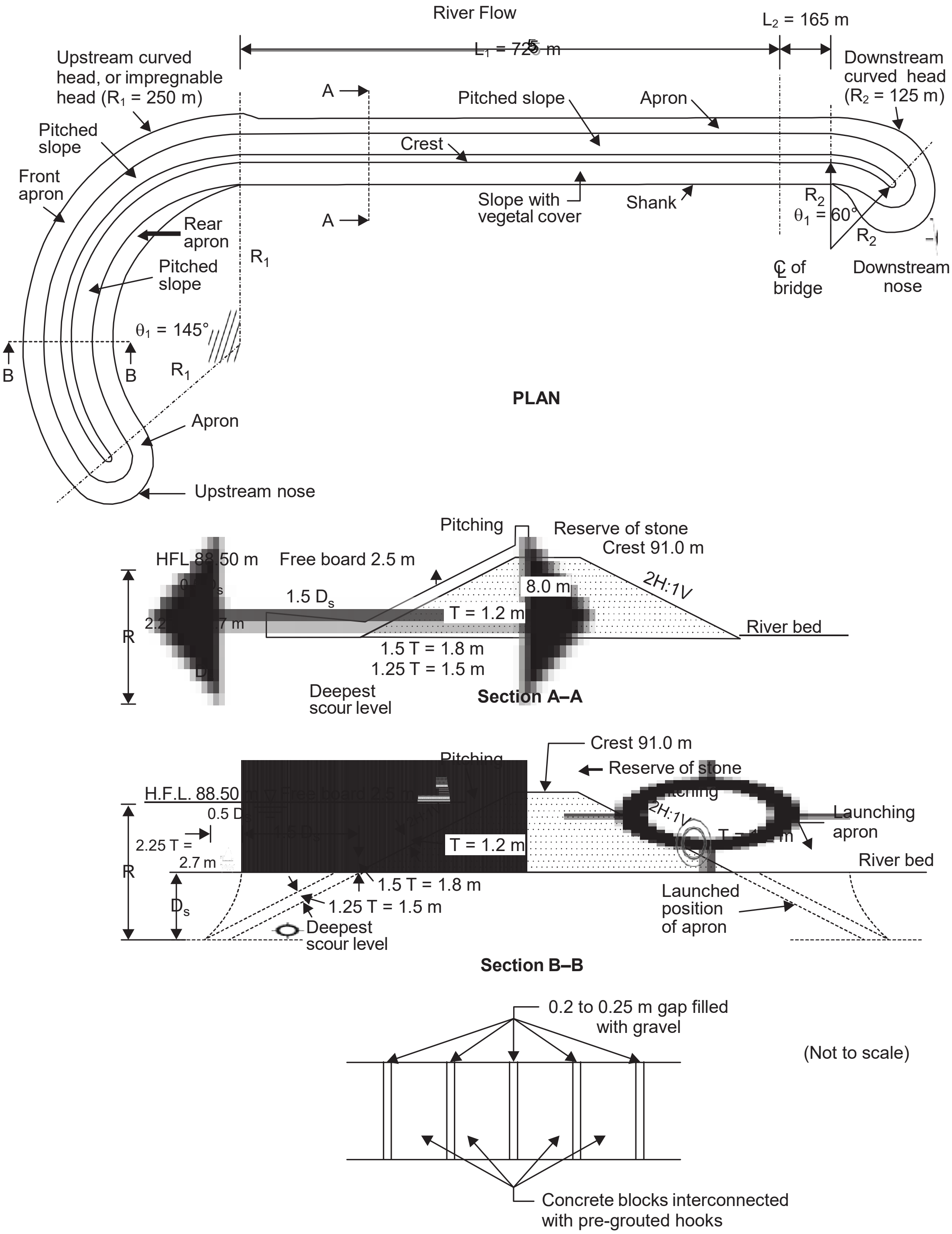
The radius of the downstream curved head, R_2 is given as

$$R_2 = 1.1\sqrt{Q} = 1.1\sqrt{13100} \cong 125 \text{ m}$$

The upstream curved head is extended to subtend an angle (θ_1) of 120° to 145° at its center, while the downstream curved head subtends an angle (θ_2) of 45° to 60°. For the present design, θ_1 and θ_2 are assigned values of 145° and 60°, respectively. Plan-form of the design guide bund is shown in Fig. 12.12.

Cross-section of the Guide Bunds :

The top width of guide bunds varies from 6.0 m to 9.0 m and the side slopes for the two faces of guide bunds are generally kept 1V:2H or flatter. A freeboard of 1.5 m to 1.8 m above the maximum flood level is provided. For the guide bunds, the top width is proposed as 8.0 m, side slopes 1V:2H and the freeboard as 2.5 m. The freeboard is kept on higher side to accommodate afflux. Cross-sections at two locations of the guide bunds are shown in Fig. 12.12.



Details of laying of concrete blocks for launching apron

Fig. 12.12 Details of guide bund (plan, cross-sections of shank and curved head)

Bank Revetment :

The thickness T required for the bank revetment is related to the high flood discharge by the following empirical equation proposed by Inglis (2, 5):

$$T = 0.04 \text{ to } 0.06\ Q^{1/3} \text{ in SI units}$$

Adopting an average value of 0.05

$$T = 0.05Q^{1/3} = 0.05\ (13100)^{1/3} \cong 1.2 \text{ m}$$

A geosynthetic filter or a conventional sand-gravel inverted filter of 0.30 m thickness may be placed on the sloping surface of the guide bunds facing the river flow and the revetment provided over this filter; see Fig. 12.13.

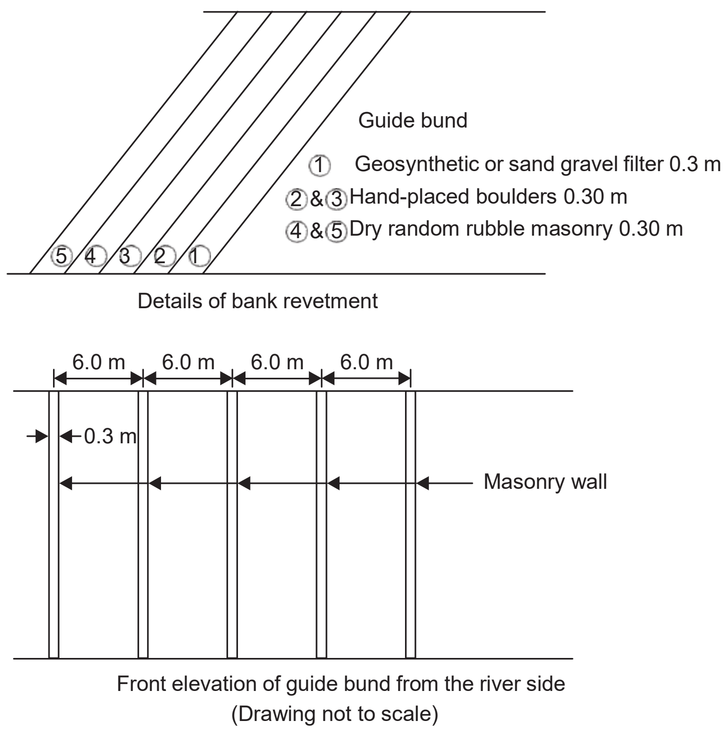


Fig. 12.13 Bank revetment

For the revetment itself, four layers of 0.30 m size boulders are suggested. The first two layers should be of about 0.30 m size hand-placed boulders are suggested. The first two layers should be of about 0.30 m size hand-placed boulders followed by third and fourth layers of thickness 0.30 m each of dry random rubble masonry on top of the hand-placed boulders. The rubble may be placed in a closely-packed formation inside a grid formed by masonry walls of 0.30 m width along the bank slope at a spacing of about 6.0 m measured in the direction of the river flow. The revetment should extend to 0.5 m above the HFL of the river.

Design of Launching Apron :

Design of launching apron requires estimation of scour depth. Discharge for the design of foundation is usually taken as 1.1 to 1.2 times maximum flood discharge. Adopting the value of 1.2Q *i.e.*, 15,720 m^3/s, the normal scour depth below HFL, R may be computed from Lacey's equation, Eq. (8.32).

$$R = 0.48\left(\frac{Q}{f}\right)^{1/3}$$

where, $f = 1.76\sqrt{d} = 1.76\sqrt{0.3} = 0.96$

Thus, $$R = 0.48\left(\frac{15720}{0.96}\right)^{1/3} = 12.19 \text{ m}$$

Actual scour depth below HFL is increased to the following values for different parts of the guide bund following the recommendations of Inglis (2):

Upstream nose : $2.25R = 2.25 \times 12.19 = 27.43$ m

Downstream nose : $1.625R = 19.81$ m

Straight portion : $1.5R = 18.29$ m

The average thickness of the launching apron in its launched position is generally taken as 1.25 T. Since $T = 1.2$ m, the required thickness of the launching apron in its launched position is 1.5 m.

The minimum size of the boulder d_{min}, to be placed in the apron (so that it is not washed away by the flow), is related to the maximum velocity U_{max} in the vicinity of the guide bund (22).

$$d_{min} = 0.023 \text{ to } 0.046\,(U_{max})^2 \text{ in SI units}$$
$$= \text{say, } 0.0345\,(U_{max})^2$$

For the estimation of U_{max}, the value of Q is taken as 15720 m^3/s and Lacey's silt factor is taken as 0.96.

Using Lacey's regime equation, the regime velocity, U, can be computed as

$$U = \left(\frac{Qf^2}{140}\right)^{1/6} = \left(\frac{(15720)(0.96)(0.96)}{140}\right)^{1/6} = 2.17 \text{ m/s}$$

Assuming the maximum velocity to be 50% higher than the regime velocity,

$$U_{max} = 1.5 \times 2.17 = 3.26 \text{ m/s}$$

$$d_{min} = 0.0345\,(U_{max})^2 = 0.0345\,(3.26)^2 = 0.37 \text{ m, say } 0.4 \text{ m}$$

The stone requirement for the launching apron is computed on the assumption of uniform apron thickness of 1.25 T in its launched position. Thus, if D_s is the depth of scour below the river bed, the quantity of stone required for launching apron of 1 m length (along the guide bund) would be $\sqrt{5}\,D_s \times 1.25T$, *i.e.,* $2.8D_sT$ m^3, *i.e.,* $3.36D_s$m^3 for T = 1.2m. This volume of stone is provided in the form of a wedge (to account for nonuniformity of stone layer thickness in launched condition) as shown in Fig. 12.12. Boulders of size ranging from 0.3 m to 0.4 m may be used to construct launching apron. Alternatively, one may provide cement concrete blocks of size 1.0 m thick, 1.5 m long and 1.5 m wide instead of the boulders. The concrete blocks are to be laid leaving a gap of about 0.2 to 0.25 m between them as shown in Fig. 12.12. This gap would be filled with gravelly material. Further, these blocks are to be interconnected with the help of pregrouted hooks to prevent dislodging of individual blocks and ensure that these blocks launch as a monolithic apron into the scour hole.

Bank Protection

Banks caving due to wave action or erosive action of river flow can lead to river breach causing large amount of losses in terms of human life, property, agriculture, and other utilities. Bank protection measures are, therefore, important to prevent bank failures.

Bank protection measures provide a shield against erosion of bank material and maintain the alignment of banks. These can be of either direct or indirect type. Indirect bank protection measures, such as spurs are not constructed directly on the bank. But, direct bank protection measures, such as revetment, riprap, etc. are constructed on the bank itself.

For providing direct bank protection, all irregularities on the bank surface are removed, and the bank is graded to an acceptable slope. The value of this slope for banks of alluvial material containing little gravel ranges between 1(V) : 5(H) to 1(V) : 6(H) below low water line and between 1(V) : 3(H) to 1(V) : 4(H) above this line (16). A layer (several centimetres thick) of coarse material, such as gravel or broken stone is spread on this slope and the chosen revetmentis laid on this layer.

Revetments are structures aligned parallel to the current and used to protect eroding banks. These revetments can be of different types such as the woven willow mattress, the framed willow mattress, the lumber mattress, the reinforced asphalt mattress, and thearticulated concrete mattress. These types of bank protection measures are usually very costly, and have been used mainly in the USA and Europe.

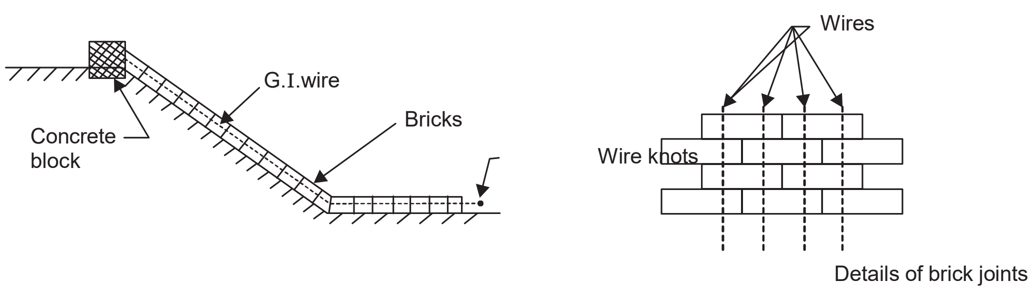


Fig. 12.14 Flexible brick pitching

A riprap paving with a toe trench is preferable to other types of revetments at sites where stone is cheap and available in plenty. Riprap of hard angular rock fragments laid on a thick layer of rubble or quarry chips is considered most durable (5). Concrete blocks can also be used when rocks are not available at reasonable costs. Triangular and tetrahedral types of concrete blocks are more suitable to resist the displacement by flowing water.

Another way of providing bank protection is by means of 'flexible brick pitching. Bricks for this purpose are manufactured such that each brick has two holes across the full width of a brick at one quarter length from either end. These bricks are then laid on the bank slope and for some distance on the river bed at the toe of the bank to serve as a falling apron. The bricks are laid in such a manner that a GI wire passes through the brick holes as shown in Fig. 12.14. The wires are knotted together at their ends and anchored in concrete blocks at their upper ends. These wires hold the bricks in place and permit small movement of bricks.

The bank revetment and launching apron, considered so useful measures for protection of banks of the alluvial streams, are not considered suitable for the protection of banks of gravel and boulder streams. In case of boulder streams, the bed does not scour much and, therefore, the apron would not be able to launch itself. Further, the concrete blocks forming the apron could be damaged and dislodged by the impact of the boulders being transported by the mountainous streams. The boulders, rolling on top of the apron, could hit the bank to cause damage to the bank. In such cases, A *RC* retaining wall near the toe of the bank may be provided from below the anticipated scour bed to about one metre above HFL. The *RC* retaining wall not only prevents the movement of its backfill but also resists bank erosion due to impact of boulders rolling along the river bed. The *RC* retaining wall would be designed for earth pressure, hydrostatic and earthquake forces, and the forces due to boulder impact.

Pitched Islands

A pitched island is an artificially created island in the river bed. It is protected by stone pitching on all sides. A pitched island is constructed with sand core and boulder lining. To protect it from scouring, a launching apron is also provided. The location, size, and shape of pitched islands are usually decided on the basis of model studies. Pitched islands serve the following purposes (3):

(i) Correcting an oblique approach upstream of weirs, barrages, and bridges by training the river to be axial,

(ii) Rectifying adverse curvature for effective sediment exclusion,

(iii) Redistributing harmful concentration of flow for relieving attack on marginal bunds, guide banks, river bends, *etc.*, and

(iv) Improving the channel for navigation.

A pitched island causes scour around it and, thus, redistributes the discharge on its two sides. Pitched islands upstream of barrages and weirs have been found to be quite effective.

Flush Bunds

Subsidiary channels of a braided river may flow very close to one of the banks of a river and result in bank erosion. Flush bund may be constructed across such channels as close to its offtake as possible. Top of the flush bund should be kept lower than the top levels of the surrounding shoal/islands to prevent outflanking of the bund on both sides when the water level in the main river rises (17).

Secondary Current Generating Structures

In addition to the above-mentioned conventional river training methods, there are other methods which depend on their capability to generate secondary currents to train a river. One such method is bandalling that has been successful in improving navigation conditions of rivers in Assam. Bandals are in the form of vertical mats or screens made of bamboos. These bandals are supported by bamboo poles that are driven into the river bed. The bandals are so erected that their upper edge is above the water surface and the lower edge extends upto about one- third to one-half the flow depth. The bandals are inclined to the stream to divert the surface currents towards the navigation channel. Bandals usually require replacement almost every year.

Surface panels consist of an assembly of deflectors suitably connected to an anchored barge. These panels cause double helical circulation stretching downstream along the river channel to be deepened for navigation purposes. Submerged bottom panels are placed on the river bed near the bank of the channel to be deepened. These panels perform rather poorly and are not as reliable as other panelling methods. Compared to the submerged bottom panels, the submerged vanes seem to be more effective.

The submerged vanes are small river training structures placed on the bed of a curved river channel. In the curved reach of a stream, the streamlines are curved and the centrifugal forces (proportional to the square of the local velocity) are exerted on river water. These forces are largest at or near the water surface. Therefore, the water near the water surface is driven towards the outer concave bank of the river and, to satisfy continuity requirements, the water near the river bed moves towards the inner convex bank. Thus, a spiral (or secondary) motion is imparted to the water flowing in a channel band. Since the sediment concentration is the highest near the river bed, the secondary flow near the river bed moves sediment towards the inner bank and deposits some of it near the inner convex bank. The sediment-deficient water coming from upper layers causes erosion of the outer convex bank. Short vertical submerged vanes, placed at suitable intervals in the outer half of the river bend and suitably inclined to the channel axis, are very effective in nullifying the secondary currents responsible for scouring of the outer convex bank of the curved channel (25). Submerged vanes, simple in design as well as construction, do not cause any significant change in either the area of channel cross-sections or the longitudinal slope of the water surface. Therefore, the stream characteristics upstream and downstream of the curved reach remain unaffected due to the submerged vanes.

The vane height above the stream bed is about 0.2 to 0.4 times the bank-full flow depth. The vanes are placed in arrays along the outer bank of the river. Each array may have two or more vanes. The vanes in an array will be spaced laterally at a distance of two to three times the vane height. The stream-wise spacing between the arrays will be about 15 to 30 times the vane height. The vanes should not be farther from the bank by more than four times the vane height.

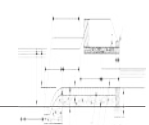
Other River Training Methods

In addition to the major river training methods discussed above, there are other types of training methods too. A reservoir can be constructed for controlling floods by storing water during a flood and releasing it after the flood has receded. Another effective and economical way of flood control is by diverting part of the flood discharge from the main river. The diverted water can flow through either a natural or an artificial river channel and ultimately either join a lake or meet the sea. Channel improvements (such as by reducing channel roughness, by dredging a channel to widen and deepen it, and by increasing discharge-carrying capacity of the river channel) enable it to pass the flood discharge at a relatively smaller stage. Soil conservation practices increase the infiltration and, hence, decrease the peak runoff.

Sills or 'bed sills' are useful in counteracting the tendency of excessive scouring (and, hence, deepening) in parts of the river cross-section. Bed sills are placed across the deepest part of the cross-section so as to partially block the flow in the deeper part of the channel. The flow near the bed is, therefore, diverted towards the shallower part of the channel. This increases the depth of flow in the shallower part. Sills are thus very useful to make non-navigable river bends navigable.

Artificial cutoffs (discussed in Sec. 12.3.3) are also useful training measures to divert the river from a curved path which might be endangering important land area.

Flooding and Inundation in Nepal Terai: Issues and Concerns



Basistha Raj Adhikari

Basistha Raj Adhikari

Abstract: During the monsoon months from June to September, all the rivers in Terai are in spate with bank-full dis- charges and cause flooding and inundation. The problems of flooding and inundation in the Terai are more critical due to change in climate in general and change in rainfall pattern/intensity in particular. This article tries to highlight the issues and concerns of flooding and inundation in the Terai and suggests measures to mitigate these issues in light of climate change adaptation.

Key words: Flooding, inundation, River Training Works, Terai, Nepal

Introduction

The Terai region of Nepal occupies about 17 percent of the land area (25,000 km^2) and forms the northern edge of the Indo-Gangetic plain. The Teraiarea is popularly known as the granary of the nation. The topography of the Terai is almost flat with gentle slope towards the south. The elevation of the Terai ranges from 65m to 300m from mean sea level with varying width of 20 km to 45 km. The climate of the Terai is subtropical with average temperature of 25ºC. The rainfall of the Terai varies from 1,200 mm to 3,000 mm per annum with occasional showers and cloudbursts. All the rivers of Nepal debouch into the Terai plain at the foot hills of the Churia and Siwalik ranges and provide water for livelihood of the Terai. During the monsoon months from June to September, all these rivers are in spate with bank-full discharges and cause flooding and inundation in several parts of the Terai. The problems of flooding and inundation in the Terai are more critical due to change in climate in general and change in the rainfall pattern/ intensity in particular. This article tries to highlight the issues and concerns of flooding and inundation in the Terai and suggests measures to mitigate these issues in light of climate change adaptation.

River Systems of Nepal

According to the origin, size and nature of the flow, Nepal's rivers are grouped into three categories:

- Large rivers originating from the Higher Himalayas,
- Medium rivers originating from the Mahabharat Hills, and
- Small rivers originating from the Siwalik and Churia Hills.

The largest four rivers of Nepal, i.e. the Koshi, the Gandaki, the Karnali and the Mahakali originate from the high Himalayas and carry large discharges. These rivers are incised in the hills and mountains and are comparatively less vulnerable until they debouch into the Terai plain. The average discharge of the Koshi, Gandaki and Karnali range between 1,350 m^3/s to 1,600 m^3/s while the average discharge of the Mahakali River is 726 m^3/s. In the Terai these rivers often cause flooding and inundation over large stretches of land adjoining their banks.

The medium rivers that originate from the Mahabharat hills experience monsoon discharge of 2,000 m^3/s to 8,000 m^3/s and create havoc of flooding and inundation in the Terai. These rivers transport significant amounts of sediment while flowing through the Churia range and exhibit lateral shifting. These rivers are wide as they enter into the Terai plain and start meandering after the Bhabar zone. These rivers are the Kankai in Jhapa, Kamala in Siraha and Dhanusha, Bagmati in Sarlahi and Rautahat, Tinau in Rupandehi, West Rapti in Dang and Banke, and Babai in Bardiya districts.

The small rivers originating from the southern slope of the Churia hills are named as Churia rivers. The length of these rivers up to the Nepal-India border ranges between 25 km to 85 km. These rivers are numerous and cause local erosion and deposition in the Terai belt. These rivers have special morphological characteristics that aggravate the flooding and inundation in the Terai of Nepal. The most vulnerable Churia Rivers are the Biring and Ratuwa in Jhapa, Bakraha and Lohendra in Morang, Sunsari in Sunsari, Khando in Saptari, Balan and Gagan in Siraha, Rato in Mahottari, Jhim and Lakhandehi in Sarlahi, Lal Bakaiya in Rautahat, Pashaha in Bara, Rohini in Rupandehi, Banganga in Kapilvastu, Khutiya in Kailali and Dhondha in Kanchanpur districts. These Churia rivers have special characteristics that are described in their proper context below.

River Morphology in the Terai

The river morphology in the Terai depends upon the origin, flow magnitude and sediment load of the river. The rivers originating from the Churia hills are characterized as flashy with negligible or low flow during the dry season. In addition, the large rivers also change their morphology as they debouch into the Terai plain with flatter slopes and wide widths. The morphological characteristics of the rivers flowing through the Terai are briefly described hereunder:

Severe Erosion in the Hill Slopes: The Churia hills are the youngest hills in the Himalayas formed about 2 million years ago (Neocene period). Their composition is made of sedimentary rocks such as mud-stones, sand-stones, and conglomerates. The Terai areas of

Nepal experience relatively higher rainfall intensity (Carson 1985) which contributes to higher soil erosion in the slopes of the Churia hills. The rivers that originate from the Churia hills bring a lot of sediment thereby eroding the hill slopes as well as river banks (Photo 1).

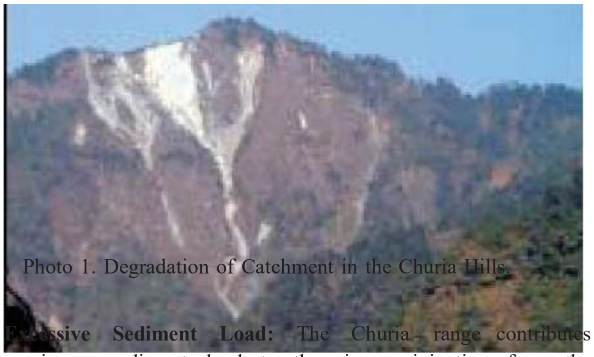


Photo 1. Degradation of Catchment in the Churia Hills.

Excessive Sediment Load: The Churia range contributes maximum sediment load to the rivers originating from the southern face (Sharma 1977). According to a JICA study, the annual sediment yield of the Lakhandehi river is estimated as high as 178,000 m^3 per year. In some stretches of the Churia rivers, almost two meters of sediment has been deposited in the last45 years (Dixit 1995). Due to the higher sediment load, the bed level of several rivers are rising significantly. The case of the Lothar river in Chitwan District is typical of this phenomenon (Photo 2).

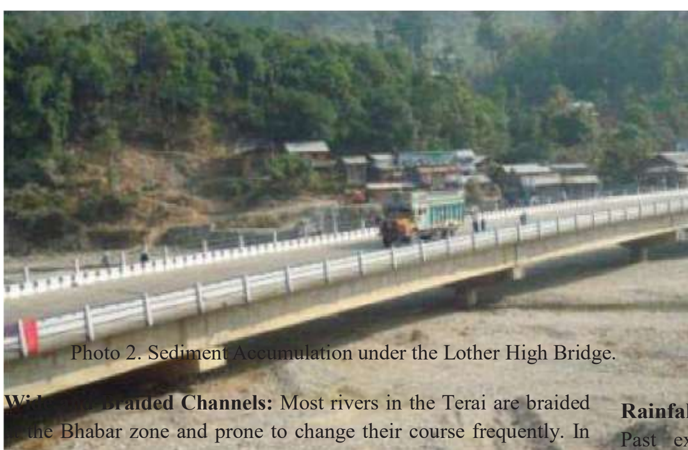


Photo 2. Sediment Accumulation under the Lother High Bridge.

Wide and Braided Channels: Most rivers in the Terai are braided at the Bhabar zone and prone to change their course frequently. In many cases, these rivers find a new path and enter into the cultivated lands leaving the old course. This is also evident in some medium sized rivers. For example, the bifurcation of the Tinau River into the Tinau and Dano rivers just downstream of the East-West Highway Bridge after 1978 flood is a typical example of the Terai river characteristics.

High Peak Floods: Runoff in Churia Rivers concentrates only in the monsoon months from June to September. Factors like short steep slope, overgrazing, deforestation, and short time of concentration produce high peak flood during the monsoon. In general, the runoff duration is less than one day and the flood hydrograph is very sharp with high peak discharge.

Photo 3. Flood in Mahakali River in 2009.

Narrow Width at Nepal-India Border: Most of theChuria Rivers have relatively narrow width at the Nepal- India border. During heavy rainfall these rivers spill overthe banks and flooding and inundation occur for some time depending on the topography, slope of the river and the waterway. A typical example of a narrowing riverwidth is the Rato River in Mahottari District. The Ratoand Jangha rivers in Mahottari District lie 6 km apart at theEast-West Highway. The span of the Highway Bridge at theRato River is 204m. After joining the Jangha and Ankushirivers, the span of the bridge over the Rato River at the Bardibas-Jaleshwor road narrows down to about 100m. Further downstream near the Nepal-Indiaborder at the Jaleshwor-Bhirttamod road, the span of the bridge is narrowed again to only 30m.

Topographical Depressions: Some of the Churia Rivers form topographical depressions such as marshy lands, swampy lands and oxbow lakes in the lower reaches of the Terai. These depressions have formed due to excessive meandering, abandoning of the old course, and avulsion of the rivers or their part. The Sunsari River at Maria Dhar, old course of the Kankai Riverare some of such examples. The medium sized river Babai in Bardiya District had also changed its course and formed topographical depressions in the west of Gularia some 80 years ago.

Rainfall-Runoff Process in the Terai

Past experience shows that flooding and inundation occur following high intensity rainfall in the Churia hillsand Terai. A rainfall intensity of 350 mm for consecutive 48 hours is considered as high intensity rainfall (Sharma 1988). In addition, rainfall exceeding 70 mm per houris considered as cloudburst rainfall (Gyawali 2011) which disrupts both the slopes and channel equilibriumat the local as well as regional scales. Precipitation events exceeding 375 mm in 24 hours have been recorded in different parts of the Terai between 1959- 1993 and maximum rainfall data is shown in Table 1.

S. No.	Station Code	Station Name	District	Mean Annual Rainfall (mm)	Max Mean Monthly Rainfall (mm)	
1	0215	Godavari	Kailali	2,279	700	July
2	0416	Nepalgunj	Banke	1,338	426	July
3	0705	Bhairahawa	Rupandehi	1,609	509	July
4	0906	Hetauda	Makwanpur	2,283	566	Aug
5	1121	Karmaiya	Sarlahi	1,718	443	Aug
6	1421	Gainde	Jhapa	2,853	683	Aug

Table 1. Maximum Rainfall Records in the Terai.
Source: Department of Hydrology and Meteorology (DHM) Data

Past Flood Events in the Terai

Historical evidence shows that the floods of 1785, 1787, 1793, 1806, 1867, and 1871 in the 19th century weremajor ones. During the 1871 floods, rainfall of 483 mm in 36 hours was recorded causing heavy loss of lives and property in Nepal's Terai and India. In 1883, about 2,300 km^2 of area situated at the northern part of the railway line in India was flooded severely. Thousands of houses collapsed and crops of some 1,000 villages were almost ruined.

In 1902, excessive rainfall along the lower reaches of the Himalaya had caused heavy floods in the Nepal Terai. In 1926, heavy floods occurred in the Bagmati River in the Terai and 75 percent of Bhadai (harvested at mid September) crops and 50 percent of Aghani (harvested in late November) crops were damaged completely in theTerai and adjoining Indian Territory. In 1934, the floods of the Bagmati River had caused it to shift its course towards the right in India. In 1935, severe floods and inundation again occurred in the Terai and India. The Sitamani railway embankment in India was breached by flood. In recent

Floods of 1978 in Butwal: With the intense rainfall in the Churia hills, a landslide occurred in the Butwal area in September 1978. The landslide had blocked the river Tinau, and upon burst of the landslide dam, the high surge of water had washed away the newly constructed bridge over the river along the East-West highway, diversion weir of the Tinau Irrigation Project and other public property. This flood had not only damaged the infrastructure and property adjacent to the river bank but also had changed the flow pattern of the river. A rainfall of 125 mm within a few hours had been experienced in Butwal (Sharma 1988).

Floods of 1987 in the Eastern region: In August 1987, an intense rainfall of 200 mm had caused flooding and inundation in the Eastern Terai. The East-West Highway was damaged in several places. In some locations, 50 cm deep water was flowing over the road pavement causing submergence of agricultural lands adjacent to the highway (Sharma 1988). The most flood affected districts then were Jhapa, Morang, Sunsari, Saptari and Udayapur.

Floods of 1993 in Central Nepal: The floods of 1993 were one of the worst in the history of Nepal which had resulted in the deaths of 1,336 persons. The maximum daily rainfall of 540 mm was recorded in Tistung of Makawanpur District on 19 July 1993. This was the highest daily maximum rainfall ever recorded in Nepal. The floods had also inundated sixty thousand ha of agricultural land in the Terai and washed way 67 irrigation systems. The peak flood of the Bagmati River is a typical example of a highly disastrous flood in the history of the country, which had washed away part of the Bagmati Barrage and inundated vast areas in Sarlahi and Rautahat districts.

Koshi Flood of 2008: The devastating Koshi flood disaster occurred on 18th August, 2008 near Kushaha village of Sunsari District due to a breach of the eastern embankment of the Koshi barrage. The barrage and embankments were constructed in the 1960s under the Nepal-India Koshi Project Agreement. Four VDCs of West Kushaha, Haripur, Shripur and Narsingha of Sunsari district were severely affected from the flood disaster. At the time of the breach, the Koshi river had only 4,700 m^3/s of water flowing through the barrage. The flood flow was minimal of August flow against the highest flood of 26,990 m^3/s (Dixit 2008) recorded in 1968.

Photo 6. Koshi Flood 2008.

Photo 7. Damage of Crops in Far-West Flood, 2008.

Far-West Flood of 2008: One month after the devastating flood of the Koshi, downpour generated by cloudbursts occurred in the Far-western region. Most of the mountain districts were affected by a series of landslides while the Terai districts were affected by floods. All the rivers that originate from the Churia hills swelled with bankful discharges, spilled over their banks, eroded adjacent agricultural lands, deposited sands and silt on nearby houses, and inundated settlements for days. The East-West highway was eroded in two-three places in Kailali and Kanchanpur districts along with damages to irrigation projects, transmission lines, and other public and private infrastructure.

Barriers as Drainage Congestion in Terai

Drainage congestion is one of the reasons causing flooding and inundation in the Terai. Some of the examples of drainage congestion are briefed hereunder:

East-West Highway: The East-West highway connects all Terai districts by a single strategic road network and is considered as a backbone of national economic development. It traverses mostly along the foot hills of the Churia range and acts as a barrier embankment especially for rivers and streams that originate from the Churia hills. In 2007 September, a small culvert near Chormara in Nawalparasi district was washed out by flood water due to drainage congestion.

Bridges and Culverts: The waterway opening of the bridges and culverts constructed in the Terai area seem to be on the non conservative side to address the floods from upstream catchments. There are several examples of damages to the bridges and culverts due to drainage congestion. In 2007 September, a highway bridge over the Dhanshar River in Rautahat District collapsed by flooding, and traffic movement was interrupted for days. Two causeways needed to be replaced in Nawalparasi district on the Danda Khola and Janga Khola. The over flanking of the East-Rapti river from its left bank upstream of the highway bridge near Hetauda by the flood of 2007 had also interrupted traffic for many days.

Urban Settlements: Most of the urban settlements in the Terai are prone to inundation after intense rainfall events due to improper drainage provisions. The inundation of Nepalgunj in 2007 July is an example of drainage congestion owing to deficient urban planning and management. The inundation lasted for a week in most of the city centers including New Road, Gharbari Tole, and Surkhet Road. A maximum rainfall of 205 mm was recorded in Nepalgunj on July 27, 2007 which caused flooding and inundation in Nepalgunj (KC 2008). Similarly, Biratnagar, Bhairahawa, Narayanghat and Janakpur are also prone to inundation time and again due to drainage congestion. The inundation of Gaur Bazar is a different case as it becomes inundated due to congestion of sheet flow over the entire area from the Lalbakeya River on the West and Bagmati River on the East. The main cause of this inundation is the construction of the Bargenia Ring Bund in Indian Territory which blocks the flow path of natural drainage from Gaur Bazar.

Hulaki Sadak: The cross-country postal road or *Hulaki Sadak* is being implemented under Indian assistance in the Nepal Terai. The proposed length of the road is

Photo 8. Mahali Sagar of Kapilvastu District.

1,446 km which is aligned mostly within 10 km of the Nepal-India border from Mechi to Mahakali. It passes through flat Terai terrain and acts a barrier to sheet flow. The confinement of the road at the location of bridges

S. No.	Name of river	Span of bridges (m)		Remarks
		E-W Highway	Hulaki Sadak	
1	Gagan	128	46	In Siraha district
2	Rato	204	102	In Mahottari district
3	Lakhandehi	250	100	In Sarlahi district
4	Lalbakeya	361	120	In Rautahat district
5	Aurahi	319	50	Aurahi and Basai form Bighi river in Dhanusha district

Table 2. Typical Bridge-spans in E-W Highway and *Hulaki Sadak.*
Source: Field Survey, 2009

may aggravate the drainage congestion. The alignment of the postal road starts from Chandragadi, Jhapa to Dewanganj, Sunsari through Rangeli in Morang District and joins the E-W highway to cross the Koshi river. In Siraha District, the *Hulaki Sadak* is just a few hundred meters north of the Nepal-India border where the Gagan River crosses it. In Nawalparasi and Rupandehi districts, it joins Bhairahawa town with Parasi and after Bhairahawa follows the Lumbini Road up to Taulihawa. The waterways provided to pass the flood under the bridges of the *Hulaki Sadak* are significantly narrow and cause flooding and inundation in several locations. In addition, the comparison of waterways of the bridges along the E-W Highway and *Hulaki Sadak* shows significant deviation (Table 2) in respect to the same river.

Inundation at Southern Border Points: All the rivers of Nepal drain to the Ganges basin crossing the Nepal- India border in the Terai plain. The total length of the border with India is estimated as 1,808 km, of which rivers act as the border between Nepal and India for about 595 km of length (Shrestha 2009). The infrastructures constructed just downstream of the border across the contour have congested the drainage passage of the natural water bodies and have caused inundation in upstream areas adjacent to the border. India has constructed dozens of the embankments, dams water control structures just at the boarder inundating Nepalese territory/farmland. Nepal's request to alleviate the problems have not been considered by India. The major structures constructed at the border points in India that cause flooding and inundation in Nepalese territory are:

- Girijapuri barrage on the Karnali river (Ghaghra in India),
- Saryu barrage on the Babai river (Saryu in India),
- Laxmanpur barrage on the Rapti river,
- Banganga barrage on the Banganga river,
- Goabari weir on the Lalbakaiya river, and
- Dheng bridge on the Bagmati river

The main canals that pass along the contour under the Koshi and Gandak Project agreements between Nepal and India have also created drainage congestions time and again. The Gandak Western Main Canal and associated structures are causing flooding and inundation in Nawalparasi District due to drainage congestion of these structures. Similarly, the Koshi Western Main Canal also adversely affects the area in Saptari District of Nepal. In addition, embankments constructed in India near the Nepal-India border block the sheet flow as well as natural drainage passage causing flooding and inundation in Nepalese territory. The major embankments constructed in India adjacent to the border are:

- Kalkalawa Bund in the Rapti river (right afflux bund of Laxmanpur barrage),
- Khurd-Rasiwal Bund (K-G Bund) in the Danav river(Kunha in India),
- Bargenia Ring Bund near Gaur Bazar in RautahatDistrict,
- Kunauli embankment in the Khando river near Trilathe in Rajbiraj District, and
- Mechi embankment in the Mechi river nearKakarbitta in Jhapa District

Furthermore, the implications of these embankment bunds are many. Lack of proper maintenance of drainage flow paths under the cross-drainage structures of irrigation canals in India also aggravates the flooding and inundation in the Nepal Terai. Drainage congestions on the Ghorasain branch canal of Gandak Project are the typical examples in this respect.

In addition, there are several isolated irrigation projects and small embankments in India near the Nepal- India border that cause flooding and inundation in the Nepal Terai. Some of the minor irrigation structures are:

- Mahali Sagar, Bajha Sagar, Siswa Sagar and Marthi Sagar in Kapilvastu District,
- Danda Barrage on the Danda river near Sunauli in Rupandehi District,
- Masauliya barrage on the Rohini river in Rupandehi District and,
- Kantawa Irrigation in the Maraha river in SirahaDistrict

The Sagars in Kapilvastu District are the typical examples of pond irrigation. These Sagars provide irrigation to the bordering area of India at the cost of inundation in Nepal territories. More than 60 percent of the reservoir area of these Sagars lies in Nepal at normal water level. The situation aggravates during the heavy monsoon rainfall and delay in gate operation by the Indian authority who live far away from the location of these gates.

Initiatives to Cope with Floods and Inundation People of the Terai area have been compelled to live withfloods and inundation since time immemorial. People have adopted indigenous local coping measures to survive the floods and inundation. The Government has also been implementing several flood protection and rehabilitationmeasures. In addition to the humanitarian support to the victims of the floods, there are institutional efforts of the Government towards mitigating flooding and inundation in the Terai, which are briefed hereunder:

Department of Water Induced Disaster Prevention(DWIDP): The DWIDP is the leading Government agency to deal with the water induced disasters in general and floods in particular. In 1991, the Disaster Prevention Technical Center (DPTC) was established to address the water induced disasters in a coordinated way under JICA assistance. Considering the positive impacts of DPTC, the Government has converted it into the DWIDP since 2000. To cope with ever increasing demands of disaster mitigation at the local level, the GON has expanded the DWIDP through divisions and sub-divisions along with the inclusion of River Training Projects known as Peoples embankment. In addition to the structural measures of river training, the DWIDP also imparts awareness and training activities to the concerned persons and institutions. The river training works carried out so far are in the Bakra River in Morang, East-Rapti River in Chitwan, and Mahakali River in Kanchanpur districts.

JICA Study on Flood Mitigation Plan: In 1999, JICA carried out the study on flood mitigation plan for selected rivers in the Terai plain. These rivers are the Khutiya in Kailali, Babai in Bardiya, West Rapti in Banke and Dang, Tinau in Rupandehi, Narayani in Chitwan and Nawalparasi, Lakhandehi in Sarlahi, Lohendra in Morang and Ratuwa in Jhapa. The study also carried out the detailed feasibility of two most vulnerable rivers, Babai and Lakhandehi, on a priority basis and suggested to implement the flood mitigation plan as soon as possible. The flood mitigation plan suggests three components for its sustainability: watershed management, river control measures and community development activities.

Extension of Embankments and Indian Cooperation

Realizing the seriousness of the flooding and inundation problems in the vicinity of the Nepal- India border, it was agreed to extend the embankments from the border along both banks of the vulnerable rivers. Since 2002, embankment constructions are being carried out in the Bagmati, Lalbakeya and Kamala rivers with grant assistance from the Government of India. Adjoining areas of these rivers in Rautahat, Sarlahi, Dhanusha and Siraha districts have been protected from the recurrent floods and inundations. However, many trans-boundary rivers are awaiting a positive response from the Indian side towards mitigating floods and inundation along the southern border.

Apart from the Governmental initiatives, there are several Non-Governmental Organizations (NGOs) working in the field of floods and inundation in the Terai.

Suggestions to Mitigate Flooding and Inundationin the Terai

Floods are natural process and we can only reduce the impacts of the floods. Following are some of the issues and concerns to be addressed to reduce the floodsimpacts.

- **Discourage the Tendency to Encroach Upon Marginal Lands at the River Banks:** Many poor people take shelter at the banks of the river due to the availability of jobs and free fertile land in the river valley. Many marginal people are maintaining their livelihoods through cultivation of steep sloppy lands and river banks which are susceptible to flooding and inundation. This tendency is increasing day by day. The case of settlements along the Tinau River is an example.

- **Manage River Bed Mining:** The increasing trend of river bed mining and its un-planned management is one of the most notorious activities responsible for flooding and inundation. On the one hand, thousands of people do find their jobs in collecting, screening and transporting river bed materials, and on the other hand, severe environmental consequences arise due to the extraction of boulders and gravel from the river bed thereby lowering its level. The problem has been aggravated due to issuance of export permits to the neighboring country by the District Development Committees (DDCs). Almost all Terai DDCs are generating revenues by exporting the river bed material and consequently inviting flooding and inundation at the homes of poor people. It is very high time to manage river bed mining in a planned, judicious and limited manner so as to sustain the environment and reduce the vulnerability to flooding and inundation.

- **Enforce Land Use Rules and Regulations:** Proper land use based on topographic and agronomic considerations is key to a planned development approach. The settlement area should be at a higher elevation than the cultivation area. Due to absence of land use regulations, several houses are constructed in the most fertile lands of the Terai. Hence, proper land use regulation and its enforcement are essential to reduce the vulnerability to flooding and inundation in the Nepal Terai.

- **Provide Adequate Drainage Passage in Road and Embankment Construction:** As mentioned earlier, development activities, mainly construction of physical infrastructure, are the major barriers to the passage of flood water. Roads in urban areas have less drainage passage way to allow the escape of incoming sheet flow. In addition, due to lack of sufficient information on hydrology, several bridges and culverts are constructed with constricted water- ways. It is necessary to provide adequate drainage passage to safely allow the the flood flows to pass.

- **Conserve the Churia Hills:** The Churia hills are the major source of sediments in the Terai Rivers. Due to degraded catchment, these hills are susceptible to erosion, landslides and mass wasting. The Churia conservation activities are being implemented on project basis since the 1990s. However, the process of degradation of catchment is not prevented. Hence, emphasis must be given to conserve the Churia hills in the long term with watershed management with peoples participation.

Conclusion

The Terai is the bread basket of the country which needs to be well safeguarded from flooding and inundation impacts. Several development initiatives need to be coordinated properly so as to provide adequate waterways for the floods. An integrated water resources management approach is the key tool to address the flood and inundation problems. Apart from watershed management in the Churia hills, land use regulations need to be promulgated and enforced to dovetail with integrated water resources management interventions.

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4. Management and Other Related Aspects

PLANNING OF WATER RESOURCE PROJECTS

GENERAL

The main aim of all water resource development is to improve the economic and environmental conditions for human living. A water resource project may serve one or more purposes and, accordingly, can be either single-purpose or multipurpose. In most cases, a project would be of a dual or multipurpose type. As such, the entire project needs to be investigated as a unit before the design requirement of a single component, such as a dam, can be finalized. A water resource project may serve one or more of the following purposes:

(i) Irrigation,
(ii) Power development,
(iii) Flood control,
(iv) Industrial water supply,
(v) Domestic and municipal water supply,
(vi) Recreation,
(vii) Fish and wild life preservation and promotion, and
(viii) Navigation.

In almost every water resource project, dam and reservoir are key components of the project. Dams impound water, divert water from a stream, or raise the water level. In exceptional cases, dams may be constructed to impound water-borne sediments and water having a damaging chemical quality. Dams contribute immensely in reducing poverty and impacts of floods and droughts besides rejuvenating rivers in dry season. Dams also enable recharge of ground water and growth of more biomass. A reservoir is a fresh-water body created or enlarged by the building of dams, barriers or excavations.

It is seldom that a water resource project consists of only a dam and reservoir facility. In a flood control project, levees and other channel control works, besides the dam and reservoir, are usually desirable. Water resource projects for power development and water supply (for irrigation, domestic, municipal, and industrial purposes) have a combination of project components to accomplish the desired objectives. Therefore, dams must be planned, designed, and constructed to operate efficiently and harmoniously with other components of the project to achieve maximum benefits at minimum cost. The economic, environmental and social feasibilities, and justification of dam must be examined in combination with those of other project components, and the total project must be evaluated and judged for its feasibility. If the evaluation of a project proposal does not show justification for its construction, it may be dropped or, alternatively, revised and updated with possible justification at a later time. A water resource project should be planned bearing in mind probable physical, economic, and environmental effects.

PHYSICAL FACTORS

Except for flood control projects, availability of sufficient water is essential for all types of water resource projects. In flood control projects, the sudden excess of water is the problem. The source of water is the surface runoff resulting from weather phenomena which are understood only in a general way. Weather conditions can be predicted only as seasonal probabilities. Weather predictions for shorter periods (a few hours or days) can, however, be made with more reliability. Historical measurements of stream flows and rainfall are considered the best available means for forecasting stream flow supplies for water resource projects.

At sites where no measurements or only a few measurements have been made, reliable correlation methods are used to estimate streamflow statistics. There is always some risk involved in building a project either too large or too small at sites of meagre stream flow measurements. In such situations, alternatives of staged development or other means of adjusting the project size and scope may have to be considered.

A flood occurring once in 100 years or less may cause enormous damage. Therefore, stream gauging records of 10, 20 or 30 years, though useful to some extent, are inadequate for flood control projects and spillway design for large dams. Besides, actual measurement of peak flood flows is difficult even if the stream is being gauged. Some other methods of estimating the magnitude of peak floods are invariably used for the planning of such works. Computation of the stream flow based on high water marks and flood channel dimensions is one such method. Alternatively, stream flow (or runoff) estimation can be based on actual measurements of amount and duration of high rainfall at rain gauge stations in the catchment area upstream of the dam site. The latter method considers factors such as principles of precipitation as affected by stream characteristics in the region, and the catchment characteristics (location, shape, vegetative cover, and geological structure). Extremely large floods are also extremely infrequent floods. Hence, the planner's judgement is crucial in deciding the size of the flood to be controlled by the project.

Two main factors which determine the site of a water resource project are the areas needing water and the location where water supply is available for development. For economic reasons, the water source must be near the place of use so as to save on cost of conveyance. Also, the source should be at higher elevation than the service area to avoid pumping. In case of projects where water is stored only for the purpose of flood control, there is no conveyance cost involved.

One can build a dam almost anywhere if one spends enough money. But, there is obvious advantage in having a dam site in a narrow section of a stream channel where sufficiently strong and impervious foundation (rock or consolidated material) is available. The abutments must be of sufficient height and be strong and impervious. Further, the dam site should not be located on or very close to an active earthquake fault. The dam site must have suitable site for spillway (a structure which releases surplus water after the reservoir has been filled up to its maximum capacity) which can be made part of the main dam only in case of a concrete dam. A dam requires a very large quantity of construction material (cement, aggregates, impervious and pervious soils, rocks, etc.) which should be available within economical hauling distance of the dam site. An easily accessible site is preferred as it involves least expenditure on communication works required for the transport of construction machinery, power house equipment construction material, and so on, to the dam site. The value of the land and property which would be submerged by the proposed reservoir should be less than the expected benefits from the project.

The area upstream of the dam site would constitute the reservoir component of the project. For economy in dam height, a reservoir site should be wide and on a mildly sloping stream in order to have a long and wide reservoir in proportion to the height of the dam. The reservoir must not be sited on excessively leaky formations. The site with the possibility of landslides, rock-slides or rock falls into the reservoir area (which reduce the storage capacity of the reservoir) must be avoided. The site should not be, as far as possible, on valuable land being used for some other purposes, such as agriculture, forestry, communication, and habitation by people. Sites with mineral deposits in and around the reservoir area should also be avoided. As far as possible, a reservoir should not be provided on a stream carrying large sediment loads which would eventually get deposited in the reservoir, thereby reducing its useful storage capacity. However, all streams carry some amount of sediment. Hence, part of the total reservoir storage is reserved for the accumulation of sediment which is likely to enter the reservoir during its intended economic life. Possibilities of constructing sedimentation basins a short distance upstream of the reservoir and/or providing catchment protection and management against sediment erosion must also be explored.

ECONOMIC CONSIDERATIONS

The cost of a water resource project includes capital investment for constructing the project facilities and the annual or recurring expenditure for operation and maintenance (including replacement) of the project. The capital cost includes the costs of planning, investigations, designs, and construction besides the cost of acquiring rights to the use of water, litigations, and rehabilitation of the affected people. The capital cost also includes the interest on the money invested during construction and up to the start of the project. The benefits likely to be received from a water resource project are widely distributed. As such, the investments on the project cannot be compared with the benefits in terms of monetary units. However, the benefits are expressed, as far as possible, in terms of monetary units and the investment and operational costs are thus compared with the benefits.

It is difficult to quantify some types of project benefits. For example, in an irrigation project, the benefits extend beyond the farmer through a chain of related activities to the people of the area. Social benefits (such as protection against loss of life by floods), recreational benefits, etc. are also difficult to estimate in monetary terms. However, benefits of municipal and industrial water services and hydroelectric power generation can be easily estimated by working out the cost of producing the same results by another reasonable alternative arrangement or by determining the market value of the product. Benefits from a flood control project can be estimated by working out the reduction in flood damages in agricultural, residential, commercial, industrial, and such other activities. The value of the land protected from floods increases and this fact should also be included in the benefits of a flood control project. Other possible benefits from a water resource project may be in the from of a fishery enhancement, water quality improvement (in downstream flows from storage releases during dry seasons), and navigation improvement on large rivers (due to storage releases during low flow seasons). Construction of a water resource project provides employment to people of the locality and is vital in areas of persistent unemployment.

Because of uncertainties involved in the estimation of project benefits, the computed benefit-cost ratio is generally not considered as the sole criterion for determining the economic viability of a project. Nevertheless, such computations do provide a logical basis for arriving at meaningful decisions on the size of the project, inclusion and exclusion of different project functions, the priority of the project, and so on. Other considerations such as social needs, repayment potential, and environmental aspects are also examined in determining the worth of a proposal for water resource development.

ENVIRONMENTAL EFFECTS

A well-planned water resource project should be desirable from economic, social as well as environmental considerations. It should, however, be noted that some of the project components, notably dams and reservoirs, cause adverse environmental effects in the regions of their direct influence. While trying to achieve major project objectives of a water resource project, the planner must examine alternative plans of dams and reservoirs to minimise adverse environmental effects.

Environment is best defined as all external conditions which affect the existence of all living beings. Different living beings affect one another, and the environmental requirements of different living beings are interrelated.

Besides, it is generally not possible to evaluate environmental effects in economic terms. In case of pollution of water and air, however, it is possible to estimate economic loss to some degree. In addition, it is difficult to assign a degree of importance to various environment conditions likely to be judged differently by different persons depending upon their own viewpoint. For example, the people of a hilly region will have a different viewpoint regarding the siting of a dam from those living in the plains where land is inundated during floods and wells go dry during drought.

The environmental effects which directly affect the livelihood and well-being of people are of prime concern. Other environmental effects on various other living beings are also of concern to man but only to the extent to which the existence of the living beings is important to man's living conditions. The beneficial environmental effects include land use improvements by irrigation, flood protection, improved water supplies for domestic and municipal uses, power supplies without consumption of fuel, water quality improvement, fishery improvement, recreational improvement, and health improvement. Various adverse environmental effects may be caused due to construction and operation of dams and reservoirs. Some of these can be mitigated by taking suitable steps while others are unavoidable. These have been tabulated in Table 14.1. The environmental check-list (Table 1.14) provides a comprehensive guide to the areas of environmental concern which should be considered in the planning, design, operation, and management of water resource projects.

Table 14.1 Adverse effects of dams and reservoirs on environment (1)

otential adverse effect	itigation method or ffect	le degree or importance of dverse effect
and use for reservoir		
oss of fish and aquatic abitat Loss of wildlife habitat oss of future access to ineral deposits	hanges of species mprove other areas for abitat None	ew species may be ess desirable than original pecies ull mitigation probably ot possible s of importance only if ineral deposits exist
oss of mountain valley reas nundation of historical or rchaeological sites nundation of exceptional eological formations	one ossibly by a useum Usually ot possible	mportant only in xtremely mountainous reas aries with each ndividual site aries with each ndividual site

lteration of downstream flows		
Reduction of fish and aquatic abitat	Maintain regulated flows	Full mitigation possible, but frequently not cceptable because of arge sacrifice of project ccomplishments
Reduction of stream flushing lows	Release occasional lushing flows	Mitigation method not proven to be worthwhile; Degree of environmental ffect depends upon specific tream situation
Changes of water quality	Selective level reservoir outlets; water aeration, if eeded	Somewhat limited xperience with selective evel outlets indicates good prospects of full mitigation
nterference with fish and wildlife migrations		
Blocking anadromous fish uns	Fish atcheries	Usually capable of full nitigation
Blocking animal nigration routes	None practical	mportance depends upon he specific site
Landscape appearance		
Excavation and waste lisposal sites	Project expenditures equired to landscape sites	Satisfactory mitigation usually possible without xcessive expenditure
Reservoir banks below naximum waterline	Minor areas may be leveloped for beaches	Degree of impact depends upon the specific reservoir ite
Abandoned construction acilities	Construction clean-up	Full mitigation possible; mportant only if not done
Erosion scars from construction roads	Principally by care of lrainage	Adverse effects can be educed but not entirely liminated with in easonable cost
Reservoir clearing waste lisposal	Controlled burning; narketing maximum mounts of wood products	Temporary effect, usually ninor, but not entirely voidable

SELECTION OF A PROJECT PLAN

Planning may be defined as the systematic consideration of a project from the original statement of purpose through the evaluation of alternatives to the final decision on a course of action. Planning of water resource project begins with some definite idea about its main purpose. It is usually economical to have a multipurpose rather than a single-purpose project. From economic considerations, the best project plan is the one for which the ratio of combined project benefits and the total project cost is maximum. The time required constructing a dam and then to first fill the reservoir before the start of the project operation is usually very large (several years) and, hence, the interest on the investment up to the start of the project operation should also be added to the investment costs. The cost of a dam and other major project features and also the benefits for at least three different sizes (the smallest, the largest and an intermediate) of the project are worked out. Using these computations, size-benefit and size-cost curves for different possible functions are prepared. A proper analysis of all this information would yield the size and functions of the project which would result in maximum benefit-cost ratio.

Generally, the needs for water services, power, and flood control in any given region continue to grow due to the increasing population. Therefore, it appears to be uneconomical to build large and costly projects far in advance of their needs. As such, physical and design conditions permitting, a project can also be constructed in stages. Because of the growing concern for environmental conditions, it is essential to take into account the environmental effects of alternative plans. Usually, there is an improvement in the environmental conditions due to the availability of water service and flood protection facilities.

However, there are some adverse environmental effects of water resource projects which affect (i) scenic beauty, and (ii) wildlife (both land and aquatic species). These effects, however, cannot be measured. A planner can, therefore, only select an alternative with more favourable or less unfavourable effects.

In making a choice of suitable alternatives, some kind of compromise is always made. These compromises may be in the form of fixing stream flows, acquisition of land to be used as wildlife tracts, siting project features to the advantage of scenic views, and providing access to areas having enjoyment potential. The following method (1) is suggested for this purpose.

Alternative plans of the proposed water resource project, having different amounts of environmental impact but accomplishing other objectives of the project, are prepared. The first step would be to make an inventory of the existing conditions of various important environmental qualities of the water resource system under consideration. These environmental factors may be ranked in order of their importance. The second step in the preparation of alternative plans would be to estimate the future environmental conditions without the project development. These conditions may be the same as the existing ones, or may be degraded or improved. The third step would be to prepare an ''optimum economic water resource project plan alternative'' without considering environmental impacts except those which are positively controlling environmental impacts. Similarly, an ''optimum environmental water resource project plan alternative'' would be prepared wherein an attempt would be made to minimise all adverse environmental impacts and still achieve some of the project objectives. If the second alternative results in significantly reduced accomplishments of the project objectives or greatly increased cost compared to that of the first alternative, the second alternative is discarded in favour of a third alternative plan. The third alternative plan would be so prepared that it would reflect a compromise between the two extreme alternatives and seek to avoid or minimise the important adverse environmental impacts while accomplishing all or most of the project objectives of the first alternative.

The role of the planner of a water resource project is to select the best of all possible alternatives. Various methods of optimization, collectively called systems analysis, are, therefore, obvious tools for this purpose. Because of large number of constraints involved, one has to often make several simplifying assumptions in order to obtain the best possible alternative. Besides, deficiencies of the input data will make determination of the true optimum a difficult task. Nevertheless, systems analysis is still the best method of determining the best possible alternative out of several feasible alternatives.

INVESTIGATIONS

The basic data, usually required for planning of dams and reservoirs, can be grouped in the following categories (1):

(i) Hydrologic data: Stream flows, flood flows, evaporation, sedimentation, water qual- ity, water rights, and tail-water curves.

(ii) Geological data: Reservoir sites, dam sites, and construction materials.

(iii) Topographic surveys: Catchments, reservoir sites, dam sites, and borrow areas.

(iv) Legal data: Water rights.

(v) Reservoir site cost data: Land acquisition, clearing, and relocations.

(vi) Environmental factors: Fish and wildlife, recreation, scenic, historical, and archaeo- logical.

(vii) Economic data: Economic base for area benefited, crop data, land classification, and market data for various purposes.

The desirable quality of these data would depend on the level of investigations. Investigations for a water resource project are generally carried out in three separate steps (or levels or stages): reconnaissance (or preliminary), feasibility, and pre-construction.

Reconnaissance (or Preliminary) Investigations

The main purpose of such investigations is to screen out the poorer alternatives and to decide the types and amounts of more expensive and time-consuming data (such as stream flow records, topographic mapping, and so on) which need to be collected for making feasibility investigations of the remaining selectable alternatives. A reconnaissance survey will identify the scope of a project plan with respect to its geographical location, project functions, and approximate size of its various components, likely problem areas, and time and cost of conducting feasibility investigations.

A complete reconnaissance investigation is, in fact, a preliminary version of a feasibility investigation carried out in a rather short time with less accuracy. It considers all the physical, engineering, economic, environmental, and social aspects related to the project. It is usually conducted with the available data. Collection of some new data, if considered necessary for reconnaissance, is made by preliminary surveys. These may include a simple cross-section (instead of detailed topography) of a stream at dam site, surface investigations of geological conditions at dam site, subsurface explorations for foundation quality at dam site, quality and quantity of available construction materials, and so forth. Preliminary designs are made by using short-cut mehods (using curves, tables, and previous experiences). Cost and benefits of the project are also estimated.

Based on the results of preliminary investigations of alternative project plans, a selection of feasible project plans is made for subsequent feasibility investigations.

Feasibility Investigation

The aim of the feasibility investigation is to ascertain the soundness and justification, or lack of these, of different alternative plans chosen after carrying out preliminary investigations. The analyses need to be of high accuracy and dependability so that the reliability of results, on the basis of which the final selection of the project plan is made, may not be questioned. It should, however, be noted that the feasibility investigation does not mean the end of the project planning. Some minor changes are always required to be made for various reasons during final designs before construction, during construction, and even during project operation.

The first step in the feasibility investigation is to collect or update the basic data of different types. The accuracy and reliability levels of these data must be consistent with the degree of accuracy required for feasibility justifications. The basic data for dams and reservoirs include topographic surveys of sites, information on stream flow and design flood, land costs, reservoir clearing costs, communication facilities, climatic conditions affecting construction, fishery and wildlife to be preserved, construction material, foundation conditions of dam site and reservoir area, availability of trained manpower, and important environmental and other considerations. The facilities and appurtenances necessary for the functioning of the project must be specified, and considered while making feasibility cost estimates.

On the basis of the feasibility investigation, a provisional selection of the site and size of the project is made. Besides, the functions of the project are also decided. The final report prepared on the basis of the feasibility investigations is submitted to the approving and funding authorities of the project.

Pre-construction Investigations

The final adoption of provisionally selected project site and its size and functions begins after the project has been approved and funded for construction. It is essential that final designs consider any new information which might have been obtained or received during the time interval between feasibility investigation and the final design. For example, an extreme low runoff season or a flood of large magnitude might have occurred during this intervening period and this may necessitate changes in the estimates of the critical dry year project water supplies or of the frequency of occurrence of a flood of given magnitude.

For pre-construction planning, more detailed and accurate topographic maps and additional geological investigations are usually necessary to reduce uncertainties about foundation conditions and construction material. This is also true of other basic data needed for planning of dams and reservoirs.

CHOICE OF DAMS

Most of the dams can be grouped into one of the following two categories:

(i) Embankment dams, and

(ii) Concrete dams.

Embankment dams include earth-fill dams and rock-fill dams. Concrete dams include gravity dams, arch dams and buttress dams. Preliminary designs and estimates will usually be required for different types of dams before one can decide the suitability or otherwise of one type of dam in comparison to other types. The cost of construction is the most important factor to be considered while making the final selection of the type of dam. Besides, the characteristics of each type of dam, as related to the physical features of the site and its adaptation to the purposes of the dam, as well as safety, and other relevant limitations are also to be considered for selecting the best type of dam for a particular site. The following are the important factors which affect the choice of the type of dam:

(i) Topography,

(ii) Geology and foundation conditions,

(iii) Material available, and

(iv) Size and location of spillway.

Topography of the site dictates the first choice of the type of dam. A concrete dam would be the obvious choice for narrow stream flowing between high and rocky abutments (i.e., deep gorges). Broad valleys in plains would suggest an embankment dam with a separate spillway.

Geological and hydrogeological characteristics of the strata which are to carry the weight of the dam determine the foundation conditions. Any type of dam can be constructed on solid rock foundations. Well-compacted gravel foundations are suitable for concrete gravity dams of small height, earth-fill, and rock-fill dams. However, effective cutoffs are required to check the foundation seepage. Silt or fine sand foundations can support concrete dams of small height and earth-fill dams. Problems of settlement, piping, and the foundation seepage are associated with this type of foundation. Non-uniform foundations containing different types of strata will usually require special treatment before any type of dam is constructed on such foundations.

If the construction materials to be used in large quantity for the construction of the dam are available in sufficient quantity within a reasonable distance from the site, the cost of the dam will be considerably reduced due to saving on transportation. If suitable soils for the construction of an earth-fill dam are locally available in nearby borrowpits, choice of an earth- fill dam would be the most economical. The availability of sand and gravel (for concrete) near the dam site would reduce the cost of a concrete dam.

Spillway is a major part of any dam and its size, type, and the natural restrictions in its location will affect the selection of the type of dam. Spillway requirements are decided by the runoff and streamflow characteristics. As such, spillway on dams across streams of large flood potential can become the dominant part of the dam and put the selection of the type of dam to a secondary position. For large spillways, it may be desirable to combine the spillway and dam into one structure. This is possible only in concrete dams. Embankment dams are based on more conservative design assumptions and, hence, spillway is generally not constructed as part of the embankment. On the other hand, excavated material from a separate spillway can be advantageously used for the construction of an embankment dam.

PLANNING OF RESERVOIRS

One major consideration in the development of any surface water resource project is the structural stability of the reservoir which should be capable of containing safely the projected volumes of water for use throughout its life time. The main factors to be considered for this are as follows (1):

(iii) Rim stability,

(iv) Water-holding capability,

(v) Loss of reservoir water,

(vi) Bank storage,

(vii) Seismicity, and

(viii) Sedimentation.

Rim stability and water-holding capability are interrelated. Rim failure can be caused due to either the sliding or the erosion of a segment of the reservoir rim. Seepage of water is mainly responsible for such failures. Major slides into a reservoir would, obviously, reduce reservoir capacity considerably. Similarly, snow avalanches and masses of ice falling from hanging glaciers can cause serious problems. Besides reducing the capacity of the reservoir, a rapidly moving slide may also generate waves. A dam may be overtopped due to the resulting wave action or rise of the water surface on account of a major slide into the reservoir. If the reservoir site is likely to be affected by the slides and cannot be abandoned, some restraining steps in reservoir operation should be taken to avoid serious failure. These steps could be in the form of limiting the filling and drawdown rates or imposing the maximum allowable water surface at a level lower than the maximum normal water surface. Alternatively, installation of drains to relieve water pressure along likely slip surfaces, some form of impervious lining, and pinning the unstable mass of its parent formation by rock bolting can be resorted to for preventing slides. Stabilisation of the unstable mass can also be achieved by strengthening or replacing weak material. Grouting is the most common remedy for strengthening such weak masses. It may be desirable to plan the steps to be taken to mitigate the effects of potential slide after it has occurred in spite of all preventive steps.

Reservoir water loss either to the atmosphere or to the ground can be a controlling factor in the selection of a site for a conservation reservoir. For a flood control reservoir, water loss is of concern only if it relates to the safety of the project. The lining of the surface through which seepage is expected is one of the preventive measures to reduce the reservoir water loss to the ground. At times, a blanket of impervious material extending from the heel of the dam is required. This too serves to control the seepage from the reservoir.

Loss of reservoir water to the atmosphere occurs due to direct evaporation from the reservoir surface. The evaporation losses are affected by the climate of the region, shape of the reservoir, wind conditions, humidity, and temperature. From considerations of evaporation, a reservoir site having a small surface area to volume ratio will be better than a saucer-shaped reservoir of equal capacity. Evaporation-retardant chemicals increase the surface tension of water by forming a monomolecular film and thus reduce evaporation.

Bank storage is the water which spreads out from a body of water, filling interstices of the surrounding earth and rock mass. This water is assumed to remain in the surrounding mass and does not continue to move to ultimately join the ground water or surface water as seepage water does. The bank storage is not mitigable. It must, however, be estimated for feasibility investigations and measured during reservoir operation for providing guidelines for reservoir regulation.

It appears that there is some effect of reservoir impoundment on the increased seismic activity of an area in which a large reservoir (having a storage capacity of more than 12 × 108 m3 behind a dam higher than 90 m) has been constructed (1). However, there have been large reservoirs without increasing the seismic activity of the region. The increased seismic activity is attributed to the changes in the normal effective stresses in the underlying rock because of the increased pore pressure. The transmission of the hydrostatic pressure through discontinuities in the underlying rock can have a triggering effect where a critical state of stress already exists. The relationship between the reservoir impoundment and the earthquake relationship is not fully understood. Hence, it is necessary that every large reservoir site be subjected to detailed geologic, geodetic, and seismic studies for feasibility decision. These observations must be continued during the reservoir operations too in order to better understand the relationship between reservoir impoundment and seismic activity.

The streams bringing water to the reservoir bring sediments too. The sediment gets deposited in the reservoir due to the reduced stream velocity. The capacity of the reservoir is reduced on account of sediment deposition in the reservoir. Usually, a portion of the reservoir storage is reserved for the storage of the sediment. The life of a reservoir is predicted on the basis of the amount of sediment delivered to it, the reservoir size, and its ability to retain the sediment. Sediment deposition at the initial stage may be beneficial in the sense that it may have the effect of a natural blanket resulting in reduced seepage loss. Measures to minimize sediment depostion in reservoirs include catchment protection through a vegetative management programme to prevent soil erosion, silt detention basins at inlets of smaller reservoirs, and low level outlets in dams to provide flushing action for removal of sediment from the reservoir. Of the various measures, the catchment protection is the most effective and also the costliest.

Integrated Water Resource Management: Principles and Applications

As per the Global Water partnership (GWP) : IWRM is a process which promotes the coordinated development and management of water, land and related resources in order to maximize the resultant economic and social welfare in an equitable manner without compromising the sustainability of vital ecosystems.

AS per the USAID: IWRM is a participatory planning and implementation process, based on sound science, which brings together stakeholder to determine how to meet society's long term needs for water and coastal resources while maintaining essential ecological services and economic benefits. It helps to protect the world's environment, foster economic growth and sustainable agricultural development, and promote democratic participation in government, and improvement human health.

IWRM cannot be seen as a blueprint or product for good water management, but rather as a paradigm with a broad set of principles, tools, and guidelines that must be tailored to the specific context of a country, region, or river basin in order to implement an efficient and effective water resource management. A basic set of principles is outlined in Box.

IWRM principles

- Integrate water and environmental management.
- Follow a systems approach.
- Full participation by all stakeholders, including workers and the community.
- Attention to the social dimensions.
- Capacity building.
- Availability of information and the capacity to use it to anticipate developments.
- Full-cost pricing complemented by targeted subsidies.
- Central government support through the creation and maintenance of an enabling environment.
- Adoption of the best existing technologies and practices.
- Reliable and sustained financing.
- Equitable allocation of water resources.
- Recognition of water as an economic good.
- Strengthening the role of women in water management

Source: IWA/UNEP (2002)

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The IWRM Paradigm

The IWRM paradigm contains important key concepts of **integration, decentralization, participation, and sustainability** (Xie2006). Due to the holistic view of the IWRM paradigm, there is a necessity for the integrated management of horizontal sectors that use or affect water resources, e.g., water supply, sanitation, agricultural use, energy generation, industrial use, or environmental protection. In addition to horizontal integration, vertical integration is also required to coordinate efforts between local, regional, national, and international water user groups and institutions (Xie2006). The main aspects regarding natural system integration and human system integration are listed in detail in the chapter annex Sect.3.13.2(GWP2000).

Besides the necessity of integration, there is also need for decentralized decision- making and responsibility at the lowest effective management level, to increase awareness for local and regional problems. Hence, IWRM seeks to strike a balance between top-down and bottom-up management. IWRM also wants to strengthen community-based organizations and water user associations.

The consideration of sustainability, as a main part of IWRM, is not only restricted to ecological sustainability for protecting the natural system, but it also covers aspects of financial and economic sustainability. This means, for instance, that resource allocation decisions have to be based on the economic value of water. Therefore, water must be priced at its full costs (Xie2006).The three key policy goals of IWRM are Equity, Ecological integrity and Efficiency, which are known as the three'E's (Postel1992):

Equity: Water is a basic need and hence there is the basic right for everybody to have access to water of adequate quantity and quality.

Ecological integrity: Water in sufficient quantities with sufficient quality should persist in the environment. Water should be used in a sustainable way, so that the future generation will be able to use it in a similar way as the present generation.

Efficiency: Water must be used with maximum possible efficiency, because of its finite and vulnerable nature. Cost recovery of the water service should be attained. Water should be priced according to its economic value.

For supporting the application of IWRM principles in practice, the Global Water Partnership(GWP) has created a tool box whose three main categories are an enabling environment, institutional roles, and management instruments (GWP2000,2004):

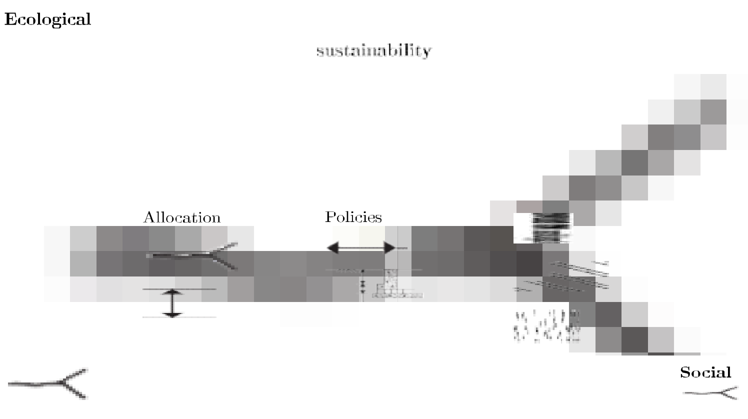
Enabling environment refers to securing the right sand assets of all stakeholders and protecting public assets. This category involves the general framework of national policies, legislation, and regulation.

The institutional roles involve the consideration of a whole range of formal rules and regulations, customs and practices, ideas and information, and interest and community group networks, which together provide the institutional framework or context within which decision-makers operate.

The management instruments include operational instruments for effective regulation, monitoring, and supporting decision-makers.

A General Framework for IWRM

For transferring the IWRM paradigm into practice, the GWP (2004) recommends an IWRM planning cycle, which is illustrated in the chapter annex. In summary, the complexity of the water cycle and interdependencies within the water sector and other sectors (e.g., food sector, electricity sector) require specific methods for integrating environmental, social, and economic issues at the level of watersheds. The paradigm of IWRM provides us with the necessary interdisciplinary tools, which come from natural water science (e.g., hydrology, geo-hydrology, and meteorology), engineering, and social sciences like political science, sociology, and economics. Often these methods, such as optimization models or decision supportive systems, etc., utilize mathematical model also necessary pre-requisite to capture complexity. Mathematically based hydro-economic models, which can be seen as a tool of IWRM, often work with simulation or optimization models and node-link networks to replicate the spatial distribution of important system elements like natural water bodies (e.g., sea, lake, aquifer, river section, etc.), artificial water bodies (e.g., canals, etc.), infrastructure (e.g., wells, dams, pipelines, pumps, purification plants, etc.), human/artificial impacts in the water system (e.g., point of use, point pollution source, non-point pollution source). Box3.2 gives an example for a numerical-based hydro-economic model, which is extensively used, among other applications, to establish an IWRM approach in California Fig.



General Framework of IWRM. Source GWP(2000)

Integrated Water Resource Management

Dublin-Rio Principles

At its birth in 1996, GWP took its guiding principles from the Dublin and Rio statements of1992.

The Dublin Statement on Water and Sustainable Development was agreed at the International Conference on Water and the Environment (ICWE), on 26-31 January 1992, a preparatory meeting of the United Nations Conference on Environment and Development(UNCED) to be held later that year.

The Dublin Statement, which included four principles on water, was submitted to the UNCEDin Rio de Janeiro, 3-14 June 1992, also known as The Earth Summit. Hence the name, Dublin-Rio principles.

GWP adapted and elaborated these principles to reflect an international understanding of the “equitable and efficient management and sustainable use of water”:

Principle 1: Water is a finite and vulnerable resource

Fresh water is a finite and vulnerable resource, essential to sustain life, development,and the environment.

Principle 2: Participatory approach

Water development and management should be based on a participatory approach,involving users, planners, and policy-makers at all levels.

Principle 3: Role of women

Women play a central part in the provision, management and safeguarding of water.

Principle 4: Social and economic value of water

Water is a public good and has a social and economic value in all its competing uses.

(See below for an elaboration of these principles.)

These principles were later summarized by GWP:

Integrated water resources management is based on the equitable and efficient management and sustainable use of water and recognises that water is an integral part of the ecosystem, a natural resource, and a social and economic good, whose quantity and quality determine the nature of its utilisation.

This summary emphasizes the importance of an integrated approach as well as clearly articulating the link between water resources management and the "3Es" of sustainable development: economic efficiency in water use, social equity, and environmental and ecological sustainability.

More on the Dublin-Rio Principles

Principle 1: Fresh water is a finite and vulnerable resource, essential to sustain life,development and the environment

This principle recognizes all the characteristics of the hydrological cycle and its interactionwith other natural resources and ecosystems.

The statement also recognizes that water is required for many different purposes, functions,and services; holistic management, therefore, has to involve consideration of the demands placed on the resources and the threats to it.

Holistic management not only involves the management of natural systems; it also necessitates coordination between the range of human activities which create the demands for water, determine land uses and generate water borne waste products.

Creating a water sensitive political economy requires coordinated policy making at all levels (from national ministries to local government or community-based institutions).

There is also a need for mechanisms which ensure that economic sector decision makers take water costs and sustainability into account when making production and consumptionchoices.

The development of an institutional framework capable of integrating human systems – economic, social and political – represents a considerable challenge.

Principle 2: Water development and management should be based on a participatory approach, involving users, planners and policy-makers at all levels

Water is a subject in which everyone is a stakeholder. Real participation only takes place when stakeholders are part of the decision making process.

This can occur directly when local communities come together to make water supply, management and use choices.

Participation also occurs if democratically elected or otherwise accountable agencies or spokespersons can represent stakeholder groups. The type of participation will depend upon the spatial scale relevant to particular water management and investment decisions and upon the nature of the political economy in which such decisions take place.

Participation requires that stakeholders at all levels of the social structure have an impact on decisions at different levels of water management. Consultative mechanisms will not allow real participation if they are merely employed to legitimize decisions already made, to defuse political opposition or to delay the implementation of measures which could adversely impinge upon a powerful interest group.

A participatory approach is the only means for achieving long lasting consensus and common agreement. However, for this to occur, stakeholders and officials from water management agencies have to recognize that the sustainability of the resource is a common problem and that all parties are going to have to sacrifice some desires for the common good.

Participation is about taking responsibility, recognizing the effect of sectoral actions on other water users and aquatic ecosystems and accepting the need for change to improve the efficiency of water use and allow the sustainable development of the resource.

Governments at all levels have the responsibility to make participation possible. This involves creating mechanisms for stakeholder consultation. Governments also have to help create participatory capacity, particularly among women and other marginalized social groups.

Principle 3: Women play a central part in the provision, management, and safeguarding of water

It is widely acknowledged that women play a key role in the collection and safeguarding of water for domestic and, in many cases, agricultural use, but have much less influence thanmen in management, problem analysis, and decision making related to water resources.
Attention to gender is essential to sound development practice and is at the heart of economic and social progress. Development cannot be maximised and sustained withoutrecognition that every policy, program and project affects women and men differently.
Addressing gender as a cross-cutting goal requires that women's views, interests and needs shape the development agenda as much as men's, and that the development agenda support progress toward more equal relations between women and men.

Gender needs should be part of the overall policy framework which can ensure that policies, programs, and projects address the differences in experiences and situations between and among women and men.

Equal participation in social and political issues involves women's equal right to articulate their needs and interests, as well as their vision of society, and to shape the decisions thataffect their lives. Their ability to do this can be strengthened through community organizations and institutions, and building participatory capacity.

Principle 4: Water is a public good and has a social and economic value in all its competinguses

Within this principle it is vital to recognize the basic right of all human beings to have accessto clean water and sanitation at an affordable price. Past failure to recognize the economic value of water has led to wasteful and environmentally damaging uses of the resource.

Managing water as an economic good is an important way of achieving efficient andequitable use, and of encouraging conservation and protection of water resources.

Value and charges are two different things. The value of water in alternative uses is important for the rational allocation of water as a scarce resource, whether by regulatory or economic means. Charging for water is applying an economic instrument to affect behaviour towards conservation and efficient water usage, to provide incentives for demand management, ensure cost recovery, and to signal consumers' willingness to pay for additional investments in water services.

Integration in IWRM

It is important to bridge components of the natural systems, like availability and quality of resources, as well as characteristics of human systems, which are fundamentally determined by resource use, waste production, and resource pollution. The main aspects regarding natural system integration and human system integration are listed in detail below (GWP 2000).

Natural system integration

- Integration of freshwater management and coastal zone management: Requirements of coastal zones have to be considered in upstream freshwater management
- Integration of land and water management: Land use influences the distribution and quality of water. Furthermore, water is a key determinant of the character of ecosystems.
- Distinction between "green water" and "blue water": Water that is directly used for biomass production and "lost" in evaporation is termed "green water", while "blue water" is the flowing water in surface and subsurface water bodies.
- Integration of surface water and groundwater management: An infiltration of water from groundwater bodies to surface water bodies and vice versa can occur.
- Integration of quantity and quality in water resources management: Aspects of generating, abating, and disposing of waste products have to be addressed.
- Integration of upstream and downstream water-related interests: Conflicts, interests, and trade-offs between upstream and downstream stakeholders using water resources have to be identified and balanced out

Human system integration

- Mainstreaming of water resources: The analysis of human activities have to involve the understanding of natural systems, its capacity, vulnerability, and limits.
- Cross-sectoral integration in national policy development: Water policy must be integrated with economic policy. The economic and social policy needs to take into account water resource implications.
- Macroeconomic effects of water developments: Water resource projects can have macroeconomic impacts (e.g., employment).
- Basic principles for integrated policy-making: Assess macroeconomic conditions of effects before realizing investment; weight expected (external) costs with (external) benefits of a policy; awareness of trade-offs in short-term and long-term
- Influencing economic sector decisions: Decisions impact water demands, availability, and quality.
- Integration of all stakeholders in the planning and decision process: Involvement of the stakeholders in the management and planning of water resources to deal with conflicting interests between stakeholders.
- Integrating water and wastewater management: Water is a reusable resource, hence wastewater flows can be a useful additional resource.

IWRM in Nepali Context

In Nepal, opportunities for economic development and people's livelihoods depend on natural resources. Water resources are regarded as the key strategic natural resource that can be the catalyst for the country's overall development and economic growth. Contrarily, the country's natural resource bases have been undergoing rapid degradation. The links between poverty, financial incentives, institutional weaknesses and degradation of water, land and forest resources are distinct and visible. Degradation of the natural resources would mean diminishing the scope of economic development. It is also established that the management of a country's natural resources demands reforms in policy, institutions and governance and the people's practices alongside investments in physical infrastructures and inputs of technology. This justifies the relevance of an "integrated" approach to managing natural resources.

Policy provisions and legislation

As Nepal has three tiers of government, the development and management of water resources falls under the jurisdiction of all three, depending on the size of the project. Besides the deep-rooted indigenous customary laws, many statutory regulations have been promulgated and amended in the country's history. Despite all this, the government was operating without any appropriate or coherent policy until the 1990s. A paradigm shift was made by the Water Resources Act 1992 and Water Resources Regulations 1993 which supported the participation of users in water development projects. Separate Electricity Act and Regulations 1993 were enacted to specific legislation for the power sector where the main thrust is hydropower development with the promotion of private sector participation. However, all these legislative measures focused on the sectoral development of water projects where the fragmented approach of sharing a particular water source between various sectors like municipal, irrigation, hydropower and others gave rise to the possibility of conflicts.

For the first time in 2002, the government worked out the Water Resources Strategy (2002-27) as a policy and strategy document for water management. The National Water Plan 2005 was brought out with detailed plans and programmes alongside the estimated costs to support the strategy. River Basin Master Plans for all major river basins in the country are at different stages of development. The Irrigation Master Plan 2019, Irrigation Policy 2013 and National Water Resource Policy 2020 are among the significant policies and plans brought out by the government. The guiding principles of all these documents reflect the common agenda that the development of the country's water resources shall be managed holistically and systematically, relying on the principles of IWRM.

Further, it is stated that water utilisation shall be sustainable while ensuring the conservation of natural resources and protecting the environment. As far as the transboundary river basins are concerned, it is foreseen that sharing of water resource benefits among co-riparian countries shall be the essential feature of water sector management. This all shows that Nepal is well set for adopting the IWRM concept in its water resources management concerning policy provisions.

Missing links

Despite all these policy provisions, hardly any water development project has been implemented following the IWRM concept. While it is essential to have policies and plans, one significant shortcoming in the past has been lack of an effective implementation strategy that has limited the outcome of Nepal's development endeavours, including water resource management. Policies as such are not legal tools. Legislative measures supporting the policy provisions are essential. For a long time, a bill to amend the current Water Resources Act 1992 (which does not talk about IWRM) is still in the making. There is no legislative provision supporting the adoption of integrated planning and implementation of water projects. IWRM is about allocating water efficiently and equitably between various competing water uses in a river basin, and the distribution of resources in itself requires strong political commitment. In the absence of legal provisions for its implementation, it is not mandatory for the agencies responsible for planning and executing the water projects to follow the principles of IWRM, which asks for a paradigm shift in the age-old conventional sector-specific planning approach.

Two other significant pillars for IWRM planning and implementation are establishing, reorganising and activating an appropriate institutional set-up equipped with sufficient and well-trained interdisciplinary personnel, and providing adequate financial resources and the essential provision of management instruments. A mechanism for the participation of stakeholders needs to be established, and the government as a whole should facilitate the process. This has been foreseen in the Water Resource Strategy. Strengthening the Water and Energy Commission Secretariat as a central planning and coordination agency of the government has also been sought in the 2005 National Water Plan. On top of this, focus must be put on the promotion of the private sector and non-governmental organizations to support the process of IWRM implementation by ensuring transparency and effective stakeholder participation.

Plans to establish a knowledge-based information system at the Water and Energy Commission Secretariat and the River Basin Offices in the three major river systems—Koshi, Narayani and Karnali—have remained long overdue. Two decades have passed since the government approved the water strategy, but these provisions are yet to be realised. The planning and implementation of water development undertakings are still being pursued following the traditional sectoral fragmented approach. Not considering a river basin as a single planning unit for water development has begun to create conflicts between local level stakeholders and the government in some planned programmers, including the proposed Kaligandaki-Tinau Inter basin Water Diversion Project.

It is time to bridge these gaps and plan and execute water development programmes holistically, leading to accomplishing the goals set in the SDGs, to which all UN member countries, including Nepal, have expressed their commitment.

Irrigation Management

For improving irrigation efficiency and subsequently management of irrigation system are grouped into following four thematic areas (or components):

- Irrigation system modernization
- Management improvement through irrigation management transfer(IMT)
- On Farm Water Management(OFWM)
- Enhancing maintenance of AMISs and ISF collection

The first two components of the proposed plan (irrigation modernization and management improvement) need to be implemented in acombined and holistic approach. Their intervention in an isolation will not be effective. However, the other two components (OFWM and maintenance support of AMISs) that focus mainly on building institution and knowledge base, need to be implemented separately as and along project (or program) . If these components are merged with the infrastructure development components, their significance could be diluted during implementation.

Irrigation system modernization

Modernization categories of FMISs

SN	ModernizationCategory	ModernizationFocus

1	Modernization of FMISswith a focus to dry seasonirrigation	Focus will be more on dry season irrigation of high value crops,rather than earlier focus on wet season supplementary irrigationof monsoon paddy. Accordingly, innovative technologicalintervention will be promoted. Examples are piped canals, onfarm reservoir, lift from adjoining rivers, semi mechanized drip /sprinklerirrigationandsoon. The WRRDC18 under the MoEWRI has already identified tens ofthousands of ha in the hills and mountains that can be irrigatedbysolar lift as oneof thetechnologies.
2	RehabilitationofFMISs	Rehabilitation will focus in increasing the efficiency of existingcanal irrigation and thereby coverage under year-roundirrigation. In FMISs, where expansion of command areas is likely,rehabilitation will require re-engineering of existinginfrastructure with appropriate technology. However, whereexpansion ofsystemareaisnotlikely,rehabilitationwillfocuson upgrading of infrastructure with a view to increase croppingintensity.
3	Bottleneck repair ofFMISs	This will focus on providing support for bottleneck repair ofexisting irrigation systems on piecemeal basis. Bottleneck repairhere refers to a lot more than the annual system maintenancethat is being performed by the local communities. Annualmaintenance is defined as that level of maintenance required tokeep the system like new. Deferred or bottleneck maintenanceis defined as rehabilitation maintenance required to bring backthesystemlike new. Bottleneck repair of irrigation system will be designed andimplemented locally by local governments at relatively lowcosts. Its main objective is to maintain the present level ofproduction, which otherwise may decline in absence of suchsupport.

Modernization of AMIS (orJMIS)

Modernization of AMISs in Nepal will have following two principal sub-components

- Undertaking deferred maintenance of irrigation system to enhance structural stability of its infrastructure
- Modernization of irrigation system to enhance irrigation delivery at all levels of irrigation system and there after over all irrigation efficiency

As the deferred maintenance of AMISs can be undertaken through a normal maintenance /rehabilitation procedure, it does not require any elaboration here.

Modernization process in a sequential order

SN	Process:irrigationmodernization	Description
1	Define projectobjective	Design of irrigation modernization will be shaped by the projectobjective. For example, if the objective is to increase productionper unit of land, a system should be able to deliver irrigation as percrop needs. While, if the objective is to achieve equitable deliveryof irrigation water, a fixed proportionate system may be designedwithlimited flexibility.
2	Design canaloperationmodality	Canal operation modality is shaped by the likely water deliveryschedule, which can be described in terms of its frequency, rateand duration at all level of irrigation systems. Accordingly, canaloperation modality19 needs to be designed based on followingconsiderations. Existingdesignofcanalsystem Agro-ecologicalsituationofthesystemarea,and Institutional capacity of water users and system managers formanagingsystemoperation.
3	Design controlstructures formodernization	Following the agreement on water delivery schedule and canaloperation modality, existing design of key irrigation infrastructure(including control structures) needs to upgraded (or redesigned) formodernization.Such infrastructureincludes: Flow control structures (regulators, dividers, outlets etc)flowmeasuringdevices Canalconfiguration(ifneeded)
4	Actualization ofmodernization	Accordingly, irrigationsystemneedstobemodernizedjointlyorinparallel with other aspects (or components) of irrigationmodernization like (a) regular maintenance, (b) OFWM, (c)institutional strengthening and capacity building, and (d)managementtransfer Subsequently, the modernized system needs to be operated as perthe designed canal operation modality, which then needs closemonitoring

Management Transfers

The main objectives of Irrigation management transfer in Nepal are:

- ✓ To relieve governments from continuing financing the maintenance of irrigation system by involving water users in irrigation management and there by creating their sense of ownership over the system so that water users start financing irrigation maintenance
- ✓ To enhance performance of irrigation system by involving irrigation users in managing their own system

- ✓ Recognizing the on going irrigation transformation worldwide and considering the outcomes of the past irrigation management transfer projects in Nepal, the master plan aims to transfer management of AMIS to two organizational entities. While doing so, management of lower level of AMISs (usually tertiary canal and below) will in variably be transferred to the block level WUA, referred to as WUG.

Thus, formation and strengthening of WUA continue to be an important part of irrigation management transfer. The management of them a in system (main and branch canals) may be transfer red to any of thef following or ganizational entities, with a view to enhance reliable bulk delivery of water to WUGs.

- Localgovernment
- Privateoperatorundermanagementcontract
- Waterusersccooperative(WUC)
- Agency-WUAjointmanagement

It is to be noted that not all these organizational entities will be suitable for undertakingmanagement of all irrigation systems. Their suitability will be shaped by the characteristics of theconcerned irrigation systems and local contexts, which needs to be examined before proceeding onmanagement transfer. Paragraphs below outline likely model of irrigation management transfer toaboveentities.

<u>Irrigation management transfer to local government</u>

- ✓ Considering the spirit of the constitution of Nepal, the public irrigation systems (or their sub-systems) whose hydraulic boundary remain within the geographical jurisdiction of local government may be transfer red to them for continuing their management. Worldwide, there are examples of local government managing irrigation system, especially the small-scale irrigation systems or sub-systems of larger systems (FAO58, 1999). Further, transfer of irrigation management to local government has been successful in Turkey and many other countries.
- ✓ In situation, where the hydraulic boundary of an irrigation system falls within the geographical jurisdiction of more than one local government, such system will be managed by coordinated approach of the concerned local governments. Depending on the needs, provincial government may play acoordinatingrole
- ✓ Weaknesses of this model are of two folds. First, irrigation system often crosses the administrative boundaries of local government. Seconds, local governments have so many other responsibilities. Despite these weaknesses, as local governments are more accountable to local water users, they can ensure that the irrigation management practices are consistent with the aspirations of the general water users.
- ✓ Further, transferring irrigation management to local government does not mean that the local governments themselves need to deliver irrigation service. Considering their other responsibilities, local governments will be free to decide management options on their own, which however need to be inline with the management transfer framework to be provided by the Department of Irrigation at thefederal level. Another important aspect is that the fundamentals of the success of IMTP rest on theestablishment of practical arrangements for financing operation and maintenance of transferred irrigation system.Local government's are in better position to meet this need.
- ✓ Management transfer to local government will be supported by modernization of irrigation system, OFWM, financing options for irrigation management, formation and strengthening of WUA, and capacity building of local government(s).

<u>Irrigationmanagementtransfertoprivateoperatorundermanagementcontract</u>

- ✓ Involvement of private sector in irrigation management, especially in the form of management contract, is not new to Nepal. In many farmers managed irrigation systems, farmers have been awarding management contract of their irrigation system to private party. The private party can be a group of people either belonging to members of a house hold or a couple of like minded people in the same village. This group under take regular maintenance of the canal, divert water from the source river to canal on daily basis, and deliver waters to farmers' field in agreed schedule and modalities. This experienceis, however, not available in the case of Nepal'slarge-scale public irrigation system.
- ✓ Scope of works of management contract will be limited to operation and maintenance of the main and secondary canal for aperiod of 3 to4 years. In general, the private operator will keep the physical system intact and will deliver irrigation water to WUGs at tertiary inlets in agreed schedule. As in thecase of other contract, the government will pay the agreed amount to the private operator for the service rendered on monthly basis or as agreed. The contract and the process will be supervised jointly by both the government (DoWRI) and the concernedWUA.
- ✓ Presently, the department of irrigation is mobilizing civil work contractors for the annual maintenance of

the main and secondary canals. Similarly, a group of short-term operators are recruitedfor operating the system during irrigation seasons. Theoretically, services for both the activities –maintenanceandoperation– arebeingprocuredoncontractualarrangement.However,thereexistnorelationship between the two service providers. The maintenance contractor is not responsible for operational difficulties due to poor maintenance. Similarly, operators do not care for likely increase in maintenance needs due to poor operation. In any hydraulic system, as both these activities–operation and maintenance – are interrelated, a private operator can come up with a cost-effective solution.Thus, in totality, the O&M cost is likely to be reduced with increased reliability of irrigation servicedeliveryintimeandspace.

- ✓ Further, as delivery of water in time and space will be one of the scope of works of the private operator, such deliveries will be systematically documented, mainly for contractual reasons. Certainly, this will be an added benefit in irrigation management. Presently, in the existing DoWRI-WUA joint management arrangements, such deliveries are not documented as there are no contractual or financial obligation senvisaged.
- ✓ Management transfer to private operator requires that such operator (or company) exist in thecountry. However, this is not the case. Although there are several professional (engineering) companies and NGOs providing agriculture related services, such irrigation operators are presently not available.
- ✓ Thus, private sector institution building will be one of the main components. Further, transferring irrigation management to private operator will involve essential structural improvements to support the management contract. As this is a new approach for Nepal, it may be implemented in trial basis in a couple of medium scale irrigation systems.

Irrigation management transfer to waterusers'cooperative(WUC)

- ✓ Water users cooperative (WUC) is one of the documented organizational entity to whom irrigation management can be transferred. FAO58(1999) notes that transfer of irrigation management to WUC is most suitable for small-scale irrigation systems or sub-systems, where management requirements are relatively simple and non-intensive.
- ✓ In Nepal, as conventional mode of DoWRI WUA joint management has not been that successful compared to the designed expectation, irrigation management transfer to WUC is being advocated. This advocacy is further supported by the provision made by the draft Irrigation Act (2015) that would allow WUAs to transform themselves to irrigation water user cooperative under Cooperative Act. Its main objectives were to ensure better access to government support and external donor funded assistance.
- ✓ The said WUCs will be the general cooperatives of irrigation water users. Unlike WUAs, which tend to be a semi formal organization with no legal authority to apply sanctions and enforce rules, irrigation cooperative will be a more formal organization that can perform both the governance and management functions directly. Politically, such WUCs are said to be stronger compared to WUAs. It is believed that WUAs often function weakly in the face of strong public bureaucracies and powerful village governments compared to WUCs. Further, WUCs can by themselves explore the possibility of external donor funded assistance.
- ✓ If one look at the status of farmers' cooperative in Nepal, they seem to be successful mainly for production industries like dairy cooperative, vegetable cooperative and soon. Irrigation being service industry, success of such cooperative in-service sector is yet to be documented. This plan suggests that irrigation cooperatives should not only look at irrigation management, it should rather have much broader responsibilities to handle and manage crop production, coordination of the use of fertilizers, marketing, transport of agricultural good sand selling to consumer. Such situation however may dilute the function of irrigation services. Further, success stories of such multi-objectives cooperative in Nepalis also yet to be documented.
- ✓ As irrigation management transfer to WUCs is a newapproach for Nepal, it may be implemented in a trial basis in a couple of medium scale irrigation systems, also on mechanized irrigation like hillpumping schemes. Further, management transfer to WUCs will be supported by modernization of irrigation system,OFWM, and institutional capacity building of WUCs.

IrrigationmanagementtransfertoDoWRI-WUAjointmanagement

- ✓ Most large scaled AMISs in Nepal are operating under the joint management model. In this model, the IMD (as DoWRI representative) manages operation and maintenance of the main system in coordination with the concerned WUA. The concerned WUA provides field level in formation on cropped area, crop type and soon to the IMD for each irrigation season.The IMD prepares canal operation plan, and operate the main system accordingly. The IMD is responsible for delivering water at each tertiary inlet (management terminal point), which is to be monitored by WUA.
- ✓ The WUA (orWUG) in turn take responsibility of managing irrigation within the tertiary command. Management activities include water allocation and distribution; coordinate with concerned farmers; maintain tertiary and field level canals; and collect ISF from farmers.

- ✓ Unlike the irrigation management transfer to the first three organizational entities noted above, the DoWRI WUA joint management mode of irrigation is considered to be very appropriate for managing large-scaled irrigation systems mainly due to their technical complexity. However, management of main system by IMD (orDoWRI) has not been that reliable and efficient due to several reasons, which in turn is influencing performance of irrigation management by WUA at lower level. As a result, sustainability of DoWRI-WUA joint management model is being questioned.
- ✓ Despite the situation depicted above, this master plan proposes to continue irrigation management transfer to the DoWRI-WUA joint management mode as one of the options. However, management modality of the main system will follow a performance oriented management with well-defined performance indicators and regular monitoring systems. The management modality will alsoinclude appropriate incentive systems for rewarding the outstanding services provided by IMD watermanagersandmembersofWUA.
- ✓ ManagementtransfertoDoWRI-WUA joint management mode will be supported by modernization of irrigation system, OFWM, and formation and strengthening of WUAs.

Formation and strengthening of WUAs for irrigation management transfer

- ✓ Irrespective of the organizational entities selected for management transfer of the main system as out lined by the fore goingsections,management of lower level of distribution system (usually a block of 100 ha or below irrigated by a tertiary canal), will invariably be continued by the concerned WUA (orWUG). In this sense, as usual, formation and strengthening of WUA will receive adequate attention. WUA is a community organization that brings together farmers for the purpose of managing a common irrigation system. In Nepal, they are formed under society registration act, and they can be registered with the government either at the office of Chief District Officer (CDO) or at concerned irrigation division.
- ✓ The hierarchies of canal networks in any irrigation systems shape the tiers of water userscommittee required to manage them. The committee at lower level of irrigation system – a blockconsisting of 100 ha or below irrigated by a tertiary canal – is usually termed as WUG, while thecommittee at the level of main system is termed as WUA. Depending on the hierarchies of canalnetwork,branch level WUA or sub-system level WUA also exist the formation of WUA should start from the lowest level to allow adequate representation of farmers at their higher level organization.
- ✓ Functions of WUA and its subsidiary committees in any irrigation system are shaped by the management transfer agreement of the concerned system. Below are some of the key functions of WUA/WUG in many IMT project.
- ✓ WUG will be responsible for all water management activities on its own with in the block. Some of these activities include water allocation, water distribution, canal maintenance, collection of ISF, resources mobilization for maintenance, and other organization activities.The WUG inturn is supposed to receive irrigation water in agreed schedule and quantum at the respective terminal points by theupper-levelserviceprovided.
- ✓ WUA and its subsidiary committee esat higher levelof canal system (aboveWUG) are responsible for (a) providing basic information to the agency (DoWRI) in designing canal operation plan; (b) support agency (DoWRI) in participatory irrigation management; (c) monitor canal operation and irrigation management activities; (d) liaison with farmers; and('e) maintain closer linkage between irrigation and agricultural development and value enhancement

Performance of WUAs

- ✓ Recognizing the fact that lack of legal authority is one of the main reasons why WUAs are notable to collect ISF from farmers, though they are authorized to do so through IMT agreement, an irrigation act was drafted in 2015, which was approved by the then Ministry of Irrigation and was placed before Parliament for its assent. However, due to changed political context (federatedstructureofgovernance), this act now needs reformulations and amendment.

The master plan proposes following strategies for enhancing performance of WUA and its subsidiary units:

- ✓ Empower WUA(WUG) legally: WUG should be the focal institution for exercising legal authority. For this, WUGs may be registered with the local government for governance support. Accordingly, local government should empower WUGs with required authority. One suchapproach is tomake WUG recommendation mandatory for any business transaction of agricultural and (for taking loan against land, purchase/selloflands, payment of land tax etc.)

  - Enhance capacity of WUAs through trainings.
  - Assist formation of robust, user-governed and well-functioning WUAs.
  - Support WUAs for enhancing coordination between agriculture, irrigation and other value

enhancement sectors at local levels.

नेपालमा सिंचाइ व्यवस्थापन हस्तान्तरण सफल नदेखिनक कारण हरु :

क) संस्थागतः

- ज.उ.स. को पर्याप्त संस्थागत क्षमताको विकास नहुनु,
- ज.उ.स. मा राजनीति. - दक्ष जनशक्तिको अभाव
- Office bearer लाई Incentives को अभाव
- दक्ष जनशक्तिलाई उपयुक्त तलब भत्ता सुविधा दिएर राख्न नसक्नु,

ख) प्राविधिक :

- नहरमा Silt जम्मा हुनु र Silt फाल्ने संरचना नहुनु,
- तल्लो तह सम्म नहर प्रणालीको विस्तार नहुनाले Water Management मा समस्या,
- पूर्वाधारका स्वामित्व हस्तान्तरण नभएको अवस्था, नसक्नु

ग) लगानी/आम्दनी :

- स-साना खण्डीकृत जग्गाहरुमा
- कृषि प्रविधि, मल विउको समस्या
- बजार र कषि व्यवसाय प्रवर्धनमा कठिनाइ
- समग्र कषिको कमजोर अवस्थामा मामा घ) सरकारी निकायको भमिकामा
- हस्तान्तरण पछी सरकारी कार्यक्रम नआउनु - जिम्मेवारी नै हस्तान्तरण भएको ठान्नु
- ज.उ.स. का सस्थागत विकासका कार्यक्रम पर्याप्त नहुनु
- अनुगमन नहुनु
- कृषि प्रविधिको विस्तार, बजार मुल्य, संयन्त्रको समस्या समाधान नहुनु

अझ स्तरयुक्त बनाउन अपनाउनु पर्ने सुधारका उपायहरु

- राजनैतिक प्रतिबद्दता,
- संरचनाहरुको राम्रो भौतिक अवस्था,
- कमी कमजोरी पहिचान गरी सुधार गरेर मात्र हस्तान्तरण,
- राम्रो तालिन सम्बन्धी कार्यक्रमहरु ज.उ.स. को क्षमताविकास (प्राविधिक, प्रशासनिक आर्थिक),
- Asset Management Plan तयार गर्ने,
- स्पष्ट लगानी योजना.
- स्पष्ट नियमहरु,
- हरेक निकायका जिम्मेवारीको स्पष्ट किटान (सरकार, कार्यालय र उपभोक्ता समिति) ज.उ.स. लाई सवल बनाउन अन्तर विभागीय सहयोग,
- कृषिका विद्यमान समस्या समाधान गरी व्यवसायीकरण र विविधिकरण गर्ने,
- जग्गा चक्लाबन्दी गर्न र खण्डिकरण रोक्न उपयुक्त नीति बनाउने,
- स-साना किसानलाई लक्षित गरी तालिम,आय आर्जन, क्षमता विकास, सिप विकास जस्ता कार्यक्रम हरु पनि संचालन गर्ने,
- संयुक्त व्यवस्थापनमा रहने प्रणाली हरुमा मूलनहरबाट निश्चित परिमाणको पानीसमयमा उपलब्ध हुने निश्चितता,
- ज.उ.स. को लेखा र खर्चप्रणालीमा पारदर्शिता,
- ISF को अधिकांश रकम प्रणाली मर्मत सुधारमा खर्च हुने सुनिश्चितता,
- ज.उ.स. लाई बहुउद्देश्यीय क्रियाकलापमा अभिप्रेरित गर्ने, आम्दनीर संगठनात्मक क्षमता बढाउने,
- सरकारी बजेट मर्मत सुधार र संचालनको प्रयोजनका लागि, भूमिका र जिम्मेवारी बाडफााड भन्दा बढी उपलब्ध नगराउने, राजनैतिक शक्तिको आडमा हस्तान्तरण पछी पनि सरकारी वजेट प्राप्त गर्दै आएका छन्, जसले कृषकलाई परनिर्भर बनाएको छ ।

OnFarmWaterManagement

- ✓ On Farm Water Management (OFWM) refers to management of water within a tertiary command with an objective of enhancing its irrigation efficiency therein, and subsequently uplift livelihood of rural community. It integrates management of main system (main and branch canals) within-field water management for crop production at farmers' field. In a tertiary command, water userscollectively manage water up to farmers' field; while management of water within farmers' field (forcrop production) is shaped by individual's interest. Thus, OFWM is shaped by both the collective and individual actions and includes multiple activities.

- ✓ Some of the common activities of the OFWM are: (i) water allocation and distribution withintertiary command, (ii) maintenance of tertiary canal and below and upgrade to pre-caste paraboliccanals, (iii)agricultural practices and water uses for crops in farmers'field,and(iv)several organizational activities within tertiary command (decision-making, resources mobilization, dispute resolution etc).
- ✓ OFWM is one of the most important components of irrigation development that links irrigationwith agriculture development. OFWM mainly focuses on building institution and knowledge base onimprovedwatermanagementandagronomictechniques.Thiscomponentshouldbeimplementedasa standalone project (or program). If merged with the infrastructure development component, itssignificancewillbedilutedduringimplementationduetoseveralpracticalreasons.
- ✓ OFWM needs to be implemented in all areas of irrigation development like AMISs, FMISs, tubewells and even private irrigation. In any area, activities of OFWM should be started with diagnostic assessment of the concerned system or subsystem that helps determine its detailed activities.Program on"OnFarmWaterManagement"will includes following activities:

  - Capacity building of agency personnel, water managers and farmers
  - Improve O&M of tertiary canals including essential infrastructure development within tertiary command, by installing precast parabolic tertiary canals
  - Land levelling and improved irrigation methods
  - Demonstration of improved OFWM and agronomic techniques
  - Infrastructure support for water augmentation: farmstorage,solar powered tubewells etc.
  - Capacity building of Engineers,WUAs and farmers
  - Improve O&M of tertiary canals and essential infrastructure

- ✓ The proposed OFWM component helps developing a site specific O&M plan for each tertiary canal in aparticipatory approach. Such plan covers all cropping cycles of a complete calendar year. The O&M plan usually includes:

  - Calibration of tertiary inlet structure for time series measurement of incoming flows
  - Design water distribution schedule and operationalize it within the tertiary command
  - Proposaltomonitoractualoperationofwaterdistributionwithintertiarycommand
  - Proposalformaintenanceoftertiarycanalandresourcesmobilization

- ✓ Preparation of participatory O&M plan also helps in identifying essential structural improvement works that are required for achieving equitable distribution of waters. Likely structural improvement worksmayinclude:

  - Field channel with intertiary command area(extentandalignment)
  - Field channel structures like division box,farm road crossing, and drainage crossing
  - Drainage channels and its structures
  - Flow measuring structure at the tertiary inlet where calibration of existing structure is not feasible

Improving ISFCollection

Causes of PoorCollection ofISF	PlanofActions
Poor service deliveryand hence little incentive to pay ISF	Enhancement of irrigation service delivery through various models of irrigation management transferred. Transparent and well-publicized regime of setting tariffs and chargeswhicharelinked tothelevel ofirrigationservice Agreed mechanisms, involving WUAs, DoWRI and local government, for monitoring irrigation service performance

	Local implementation (of nationally agreed formulas) to ensure localconditionsreflectedin irrigationfees Ensure equitable water distribution to encourage all WUA members topay
Lack of legal authorization to WUA for enforcing rules of irrigation management and ISFcollection	Register WUAs with local government for governancesupport Establish bye-laws and other supporting legislation/rules to preserve principles of payment for irrigation service, and support WUAs in institutionalizing irrigation rules and concept of"service–for– fee"
Inadequate capacity of WUAs(orWUGs)	Enhance capacity of WUAs(orWUGs) for all aspects of ISF collection
Payment avoidanceby some influential members	Define the legal basis of WUAs for setting fees in relation to service andits collection A graduated system of sanctions should be in place to oblige paymentsLegal action to collect.
Lack of incentive topayISF	Increase the incentive to pay by ensuring adequate levels of serviceprovision At the farm level, have clear agreement about the set of exceptional circumstances under which ISF may be waved or reduced
Current system offee collection involves high costand low collection efficiency	Involve WUAs, with local knowledge and presence, in fee collection onanincentive-earningbasis Create legal and procedural basis for delegated fee collection byWUAs/local government agencies Allow partial retention of fee-by-fee collectors to provide incentives for improved collection

Participatory Irrigation Management

Participatory irrigation management

- The term participatory irrigation management (PIM) refers to the participation of irrigation users, i.e., farmers, in the management of irrigation systems not merely at the tertiary levelof management but spanning the entire system.
- Participation should not be construed as consultation alone. The concept of PIM refers to management by irrigation users at all levels of the system and in all aspects of management. This is the simplicity and flexibility of PIM.
- There can be different forms of participation at different levels in the system with

varying degrees of accountability and responsibility.

- Management by irrigation users, rather than by a government agency, is often the best solution.
- Contrary to the traditional concept that irrigation management requires a strong public- sector role, the PIM approach starts with the assumption that the irrigation users themselves are best suited to manage their own water.
- "Participation in irrigation management involves a larger role for farmers, water groups, andother stakeholders.
- It may range from offering information and opinions during consultations, to fully enabling farmers to act as principal decision makers in all or most project activities.
- There have been increasing efforts to use participation in various forms to improve the quality, effectiveness, and sustainability of irrigation systems.
- This makes it important to learn what has and has not been achieved in efforts to improve participation in irrigation management.
- Farmers' participation in irrigation management is not entirely new to India. There is considerable evidence that farmers in pre-independence years had been involved in irrigation management in different parts of the country.
- The phad system of Nasik and Dhule districts and the Malgujari tanks of Chandrapur and Bhandara districts in Maharashtra, the Ahar-Pyne system of Bihar, the Kuhl system of H.P. and the Kudimaramath of Tamilnadu are some of the important examples of PIM under traditional irrigation.
- Vestiges of these practices still survive though these have become quite weak or even extinctwith the passage of time.
- A few formal water users associations were also formed from time to time like the Vadakku Kodai Melazhahian Channel Land Holders Association in Tamilnadu in December 1959, Malinagar Irrigators' Water Cooperative Society in Maharashtra in 1967, Vaishali Area Small Farmers Association in Bihar in 1971, Mohini Water Cooperative Society in Gujarat in 1978.
- These were, however, isolated examples which could be counted on fingers. Irrigation management from top to bottom remained concentrated in the hands of the government.

- It may be said that since 1972, after the establishment of CADA, a large number of farmer organisations at the outlet level were formed under the CAD projects.
- These were variously described as pipe committees, outlet committees and WUAs. These, however, lacked authority and responsibility and, therefore, could not serve any useful purpose. Many of these became non-functional after some time.

Irrigation Management Transfer: Strategies, Process and Outcome

Background:

Parallel to the concern about natural resources management, two other major movements have been emerging across the globe and shaping policy: (i) liberalism; and(ii) a call for a more participatory development approach.

The former is centeredon the idea that in order for countries to move forward – to progress – they should *inter alia*:

- open their economies to competition;
- remove trade barriers;
- open markets;
- deregulate;
- eliminate subsidies;
- privatize their industries;
- diversify providers of goods and services;
- Expand their commercial frontiers based on the principle of comparative advantage.

The participatory movement has advocated that the size of government should be reduced and that people should participate more in governance, management and financing resource development in order to promote sustainable and equitable development. Participation promotes the subsidiary principle of making decisions at the lowest level possible, thereby increasing stakeholder participation.

Towards the end of the twentieth century, many developing countries were moving in the direction of major change in their economic policies, including reductions in the size and budgets of government. Pressure was mounting on the agriculture sector to become more efficient. Many governments made efforts to collect irrigation service fees but few were successful. The time for more basic change in the irrigation sub sector was ripe. One such reform, IMT, was emerging worldwide. The philosophy behind IMT lies in the perception that increased ownership, decision-making authority, and active participation in the operation and maintenance (O&M) of irrigation systems would create or force a binding commitment from water users to be more effective and responsible towards their obligations. If farmers were to assume the costs of running the irrigation systems, the incentives to succeed in their management were bound to increase. This is the principle of subsidiary, or that decisions are made at the lowest level possible, a pillar of what is now perceived as “good” water governance. On the other hand, governmental irrigation agencies (usually constrained by bureaucratic procedures, dwindling budgets and rigid policies) became inefficient and had unmotivated personnel and low system performance. Therefore, IMT emerged in response to the need for sector reform, the merits of self-sufficiency, and the drive for increased participation of water users in irrigation system management.

Definition

The term 'irrigation management transfer' means the relocation of responsibility and authority for irrigation management from government agencies to non-governmental organizations, such as water users' associations. It may include all or partial transfer of management functions. It may include full or only partial authority. It may be implemented at sub-system levels, such as distributaries canal commands, or for entire irrigation systems or tube well commands.

The term 'participatory irrigation management' normally refers to the involvement of water users in irrigation management, along with the government. It is not the same as IMT - which is about replacing government, not just working with it. After transfer, the new service may or may not be provided directly by a farmer organization. The service provider may be a financially autonomous utility, semi-municipal water district, mutual company or other local entity. But it will normally be governed, at least in part, by the farmers, who are the primary users of the service. IMT is further distinguished from decentralization, which normally refers to the movement of decision-making authority to regional or local levels from a central authority - but still within the same government organization. IMT is the transfer of responsibility for irrigation management from one organization to another.
IMT is a multi-faceted reform which may involve changes in:

- Public policy and legislation;
- Mandates and structure of public and local organizations;
- Agency budgets, personnel policies and assignments;
- Water rights and farmer organizations;
- Operational procedures and technology design;
- Installation of new support services; and more

The overall objective of the Irrigation Management transfer of the selected Agency Managed Irrigation Systems is to improve service performance and service delivery of the selected AMIS where management transfer to WUAs would be completed and consolidated.
It comprises:

- Completion/consolidation of Management Transfer Plan including streamlining and strengthening of WUAs,
- Essential structural improvements,
- Repair or procurement of buildings, transportation, communication, maintenance and information technology equipment and
- Capacity building of WUAs and DOI.

There are four different phases during the implementation: (IWRMP, DOI)

1. Preparation of the transfer that ends up with the signature of the transfer agreement in a SCC meeting. During this period local NGOs can provide services as social mobilizes. A working capital could be provided through the project to the WUAs for establishing the

office and administrative cost in accordance with the designed mode and planned objectives.
2. Consolidation consists of the 3 year period when ESI activities and other commitments stated in the legal transfer agreement are programmed to be met. Construction units for ESI works need to be established and arrangement for procurement of equipment, training and M&E programs need to be provided.
3. Reformed operation when the irrigation system operates in a new normal state with DoI scheme local offices operating with redefined structure and responsibilities. No permanent staff should be hired that increases the overhead for reformed operation.
4. Implementation of essential structural improvement of the secondary and tertiary canals should be done with WUAs involved actively in the management of the works and part of the works being contract out to them as per GON regulations. Investments in ESI at the branch association level will be suspended if the previous year agreed contributions are not met by WUAs.

IWRMP – Irrigation management Transfer Program (Component B)

The overall objective of this component is to improve service performance and service delivery of selected public irrigation schemes in the Terai where management to WUAs will be completed and consolidated. The component is designed to address the problem in large public irrigation schemes (AMIS or agency –managed irrigation systems) of below –capacity performance, poor O&M, negligible cost recovery (below 5 percent on average) and inadequate maintenance funds.

The component is to provide improved arrangements and instruments for O&M to AMIS for empowering WUAs to operate, maintain and manage parts of the irrigation systems for their sustainability. The management transfer to the WUAs would mean turning over the governance, management, and maintenance responsibilities. The DoI will operate and maintain the headwork's, de-silting basins and, in some cases, main canals and head regulators. The WUAs will operate and maintain the transferred systems and related assets as per the legal transfer agreement guidelines.

The expected outputs from this component are:

- (i) efficient and equitable service delivery by financially and institutionally sustainable WUAs;
- (ii) improved physical performance of the selected irrigation schemes; and
- (iii) Reliable bulk water service delivery by DoI in line with the Transfer Agreement. Other outputs include :
- (iv) formation and strengthening of WUAs to become self-governing, self –financing, and self-regulating organization;
- (v) improved arrangements and instruments for O&M of public irrigation schemes and
- (vi) Completion and consolidation of water management transfer of 9 schemes (24 sub-systems) in four irrigation systems to Water User Associations by which they will be assume full responsibility for equitable water distribution, collection of ISF from beneficiaries, and operate and maintain the respective parts of the systems on their own.

Component B aims management improvement with the legally empowered WUAs taking the responsibility of existing 9 schemes (24 sub-systems) of 4 AMISs. This systems/sub-systems was selected on the basis of condition of infrastructure, receptive user organizations and relatively favorable socio-political environment. The four AMIS systems are: Kankai, Sunsari Morang, Narayani, and Mahakali. These system/sub-systems will cover about 61,000 ha.

The approach of the component is to improve performance and service and service of the selected schemes through focus on sustainable governance mechanisms, financing arrangements and capacity building of WUAs and DoI in addition essential structural improvements.

The following selection criteria are used for sub-project selection.

- Availability of water
- Willingness to change
- Less Conflict
- Local capacity of DoI field staff
- Reasonably functional Status of infrastructure
- Local capacity of the existing WUAs

In order to implement the Irrigation Management Transfer component B, the following activities are designed to make the management transfer a successful endeavor.

- Preparation of Management Transfer: Survey and Investigation for the preparatory works, preparation of Asset and Financial Management Plan,
- Civil Works for Essential Structure Improvement,
- Capacity buildings of WUA,
- Consulting services for IMT, and
- Logistic support
- Equipment and Tools
- Establishment Grants to WUA for three years and incentives to field staff of DOI.

FARMER MANAGED IRRIGATION SYSTEMS IN NEPAL AT THE CROSSROAD**

Prachanda Pradhan*

Abstract

In Nepal, Farmer Managed Irrigation Systems (FMIS) occupy special status in the national economy and food security system. Out of irrigated area in Nepal, almost 70% fall under farmer managed irrigation systems. They are the vibrant systems. The history of FMIS is long and they are still active institutions in Nepal. Hence, FMIS are the national heritage of Nepal. Secondly, FMIS are the symbol of democratic values. The community owning the systems manages the resources on their own. They evolve the rules and regulations on their own and implement them with consensus within the community. Hence, FMIS has a special place in irrigated agriculture in Nepal.

The irrigation sector in Nepal is facing new challenges. FMIS is not exception. FMIS is facing the challenges brought by population growth, pressure for increased demand on food, environmental degradation and unavailability of local construction materials and competition on the allocation of water.

FMIS is at the crossroad. There are both internal and external challenges to FMIS. The internal challenges are of design , of construction materials due to the depletion of the local construction materials, competition on the use of water, stagnated economic development, new legislation either ignored the existence FMIS or attempt is made to bring these systems under the control of local administration ignoring the need for development of polycentric system to strengthen the democratic values at the grassroots level and the process of assistance by the government to FMIS.

OVERVIEW OF IRRIGATION DEVELOPMENT:

There have been changes in the irrigation management over period of time. In 1960s, the increase in agriculture production was conceived by more investment in the irrigation infrastructure development. Around 1980s, it was found that irrigation infrastructures being built over period of time have been deteriorated. It is recognized thatthe participation of the beneficiaries is import for the better maintenance and management of the irrigation systems so there has been promotion of participatory irrigation management.

Irrigation has traditionally consumed a large proportion of the world's water. At the beginning of the century, 90% of water use in the world was for irrigation. By 1960, itwas about 60% (Biwas 1993). In defense of this water use, Wallingford (1997) pointed out that irrigated agriculture produced 40% of food and agriculture commodities from 17% agriculture land. This makes food security critically dependent on irrigation. The dependence is most critical for Asia where 60% of food production is from irrigated lands. Similarly, long term impact has been felt in irrigation sector in Nepal.

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Many changes have been taking place. There has been increase in population growth. This has put more pressure for the increased demand on food. This situation puts more pressure in the irrigated agriculture. In 1990s it is recognized that water is a scarce resource and it will continue to be a scarce resource so increase of agriculture production per unit of water has to be increased. Hence, this new situation also puts pressure in the management of irrigation system for irrigated agriculture. Multiple use of water has increased so the same source is in competition with drinking water, irrigation and small hydropower. Drying of source of water of these systems due to depletion of forest coverage has contributed in the hardship of water availability. Climatic change has also contributed in the shortage of water for the use by the people.

IRRIGATION MANAGEMENT AS SOCIO-INSTITUTIONAL AND TECHNICAL ISSUE

Irrigation management is not only one-dimensional activity. It has multi- dimensional activities. They include managing organizations, which operate and deliver water. It also deals with farmer's organization, agriculture credit, extension services and market conditions and water right issues. Hence, irrigation management is to be seen as social, institutional and technical activities. It is no longer considered irrigation management only as technical problem. Changes in irrigation management mean the establishment of multi-disciplinary irrigation department open to the farmer's participation in irrigation management. The irrigation management changes also have to respond to the irrigated agriculture and increasing productivity per unit of water.

BACKGROUND OF FARMER MANAGED IRRIGATION SYSTEM IN NEPAL

Nepal portrays a rich tradition of community efforts in natural resource management especially in water resources, forestry, and pastures. Customary norms have delineated water as community resource with elaborate usufructory rights and communitygovernance structures for the management and utilization of these resources by village societies. Apart from these community-based values and norms, state policies and practices have historically been conducive to reinforced community roles in natural resource management. The edict of King Ram Shah in the 17th century mandated water resources related conflicts to be settled at the community level itself. Though such mediation had to take into account local power structures, it nevertheless allowed community initiatives and governance structures to evolve. In Nepal, over 70 percent of the irrigated agriculture is undertaken through farmer managed irrigation systems.

Both forestry and water resources in Nepal have been subject to various policies and programs spun by the government and multiple donors. In the irrigation sector, only as recent at the 1970s did farmer managed irrigation systems (FMIS) gain recognition within the plans and policies of the state. Truly these FMISs have contributed to food and water security of the nation based primarily on community efforts mediated by their own power relations.

Nepalese farmers have, by and large, recognized the importance of water resources for centuries and have been constructing irrigation systems at their own initiative to intensify their agriculture production. Irrigation development in the country remained in the hands of the people for many years. This tradition has given birth to the FMISs scattered all over Nepal. These systems have developed their own rules, norms and procedures of management.

In the FMISs, farmers are responsible for all management activities, encompassing water acquisition from the source to delivery to the plant in the field and management of the system including the resource mobilization and management ofresources for O&M. In most of the systems, the extent of the need for resource mobilization for O&M of the irrigation systems have influenced the structure of the organization.

In Nepal, FMIS occupies special status in the national economy and food security system. It is estimated that 40% of food production is produced out of 15,000 FMIS in hill areas and 1700 systems in the tarai of Nepal. Out of the irrigated area in Nepal, almost 70% fall under the FMIS. They are the vibrant systems. FMIS have long history and they are still active institutions in Nepal. Hence, FMIS are the national heritages like other national monuments of Nepal. Again, FMIS are the symbol of democratic values. The community owning the system manages the resources on their own. Hence, FMIShas special place in the irrigated agriculture in Nepal.

Farmers have developed their own irrigation systems taking account of geographical impediments and limited services from the government in the past. They have managed their systems by adjusting the operation to the soils, climate, topography and social structure of the particular location over a period of many years. These environmental conditions, which vary tremendously throughout Nepal, have contributed to different patterns of irrigation organization. In addition to distinctively different organizational patterns for the well defined tasks of water acquisition, allocation, and distribution, methods of system O&M, and organizational activities regarding conflict management, communication, resource mobilization and decision making vary. The various patterns of organization are also related to the physical type of irrigation system: hill, river valley, or Tarai system. (Pradhan, 1989).

ORGANIZATIONAL BASIS OF FMIS FOR RESOURCE MOBILIZATION:

The irrigation organization in FMIS evolved on its own without any external assistance. Hence, these organizations are indigenous ones, which evolved over period of time to manage the natural resources within their environment. During the evolution of these organizations, water distribution principles, water share, water rights, obligations, and resource mobilization basis were evolved. The members of the FMIS water usersassociations internalized these principles.

No single factor or element brings water users together in an irrigation organization. Different systems have different elements, which arise as the prominent feature. Water right issues, resource mobilization, water distribution, a sense of belonging to the community, preservation of an individual's water right are different unifying factors. However, it is not necessary to have all these features present for an irrigation organization to function. In the following section, three prominent bases for resource mobilization for the management of these systems that are found across FMISs are described.

(1) Water as community property: an organizing force to mobilize resources forO&M.

The dynamics of the functioning of FMISs can be better understood from the perspective of common property resource management. Valuing water, as "Community Property" can become the organizing and unifying force for farmers in a given system. The effectiveness of an irrigator's organization can be placed on a continuum, ranging from anarchic to well organized depending on the collective interest in irrigation water. Non-compliance with rules for water acquisition, allocation and distribution, and resource mobilization results in "anarchic" application of irrigation water, where individual interest prevails over collective interest. In an well-organized system, irrigation-related tasks are performed collectively by the beneficiaries, or all individuals carry out group-agreements.

Anarchy in an irrigation system results where group norms and values are not observed. Water is then considered as a resource to be extracted for individual benefit on the basis of "might is right". In an anarchic situation, water allocation, acquisition, distribution and conflict resolution depend on individuals settling problems with other individuals. Generally, the more powerful and influential individuals are able to extract a larger share than others are.

In an well-organized system, the acquisition of irrigation water and its application for agriculture use are based on community decisions. Committee members are elected or selected to manage the system on behalf of the community and are accountable to it.

Water acquisition is usually a collective effort, i.e. the community pools itsresources either in the form of cash or kind or labor to do this. The allocation principle is also decided collectively by the irrigator community. The distribution of water according to the criteria prescribed by the irrigator community is an effort to distribute the community resource for individual use. Limits are placed on the extent to which individuals are allowed to use these resources. Hence, water allocation and distribution become transparent in FMIS. In Agency-managed systems, water allocation is not usually transparent.

If some one violates the norms of allocation or distribution by "stealing" water or depriving others of the share of water assigned to them by the community, he is subject to punishment. A penalty is imposed depending on the gravity of the offense and according to the norms and values of the system. The irrigator community determines the terms of the penalty. This is intended to prevent an individual from extracting more resources thanallocated by the community.

Within FMIS collective decision-making, transparency and accountability are institutionalized. The executive committees of the FMISs usually are accountable to the general assembly of the irrigators association. So, collective decision-making process is institutionalized in FMIS.

(2) Operation and Maintenance Cost of the System

Operation and Maintenance have similar implication like water distribution and allocation. The collective contribution of the community is the basis for the mobilization of the resources from the members of the community. The important point to understand is that the rate of resource mobilization in a FMIS. It is usually found that the contribution for the resource mobilization is made from all members of the beneficiary group. In case of defaulters, the community takes the responsibility of realization of thoseresources required for O&M the amount required for O&M will be agreed by the collective decision of the members of the irrigation association. The resources would consist of labor, cash and materials.

It is oftentimes misunderstood that the labor contribution for the system is voluntary or cash contribution is voluntary. However, it is not true. It is not voluntary at all. The contribution is the part of obligation of the members towards the system against the benefit and resources to be derived by the members from the system. Hence, the common property resource management like water includes both rights of the members in terms of the water right as well as the obligations towards the system in order to ensure the continuity of the right over the system.

The O&M cost per hectare in hill, river valley, and Terai systems are different. Hill systems have to mobilize between NRs. 400-NRs, 535/ha as compared to about NRs. 100/ha for river valley systems while Terai systems spend about NRs. 270-572.

In hill irrigation systems, conveying the water from the source to the command area is the aspect of operation and maintenance requiring the greatest effort by the users. The distance from the intake to the command area is usually long, passing through steep, rocky terrain prone to frequent landslides. This requires frequent repair and great amounts of labor each season.

Terai irrigation systems usually have large command areas and use large rivers as their source of irrigation water. Floods in these big rivers wash away the intakes and require frequent repair to sustain a supply of irrigation water. Hence, in the Terai,maintenance of the intake is the largest component of costs.

River valley systems have lower O&M costs because their command areas are close to the water source and no long conveyance structures are required. The terrain is not difficult and fewer repairs are necessary. The important factor to take into consideration in respect to government subsidy is that there is no government subsidy in O&M of FMIS as against the high level of subsidy in agency managed system. The sustainability of these systems in respect to O&M is not in question.

Two O&M cost tables are given in Annexes, which show the cost of system management and its return to the farmers.

It is made clear that the O&M cost are born by the farmers themselves. Compared to agency managed system, FMIS does not have to provide subsides for FMIS O&M.

(3) WUAs in FMIS as Instrument for Resource Management:

The WUA functioning is important feature of FMIS so it is important to identify the factors contributing for effective Water Users Associations in FMIS in Nepal. Following factors are the general observations among FMIS Water User Association. These factors make WUA as an effective instrument for resource management

1. Wider participation of the members of the system and equal distribution of stake among head, middle and tail end farmers make the organization strong.

2. Mutual dependence between head and tail farmers due to difficulty of water acquisition or resource mobilization make the farmers respect each other. In such system, benefit would be equally distributed. This feature makes the WUA to stay together.

3. Transparency of irrigation related activities are important. This takes place in the annual general assembly meeting of the WUA. During this time, rules and regulations and statement of income and expenditures would be discussed. The elected members of the WUA would be accountable to the general assembly. The participation in the general assembly would make the members know about the system. Under such system, water rights are made transparent.

4. Resource mobilization is one of the major activities of the WUA. Resource mobilization based on equality is important. Cash, kind or labors are to be recorded properly. It should be transparent and account is open to all members of the system for inspection.

5. Water would be considered as the community resource so the rules for water distribution is agreed by all members. Decision for water distribution is to be made collectively and enforced by the committee. There are provisions of punishment for not complying the water distribution rules. These provisions make the WUA work and be effective.

6. Water right is usually specified and it is linked with the obligations and resource mobilization.

7. The legitimate executive committee formed on the basis of the voice of the member farmers would be effective one. It can act on behalf of the assembly of WUA. This gives room for wider representation of the farmers in the executive committee.

8. The general assembly would be effective one. It meets at least two times a year. Overall rules and regulations are to be passed by this assembly. Each year, it reviews the situation and comes out appropriate rules and regulations for the management of the irrigation system.

9. The executive committee should be accountable to the general body.

These are the general features of effective WUA. However, WUA are influenced by quantity of water availability, water acquisition procedure, and water right and distribution system. In most of the FMIS, water is taken as community resource and allocation and distribution of water would be done by the collective decision of the irrigator's community. The defaulters would be punished by the collective decision of thecommunity.

His Majesty's Government of Nepal has been providing assistance to FMIS for physical rehabilitation along with support to institutional capacity development. In rehabilitation of FMIS, physical infrastructure took priority over the institutional capacity development. It is often considered the physical improvement is separate from institutional capacity development. As the result of it, dysfunctional organizations have surfaced resulting adverse impact on resource allocation and distribution, resource mobilization and agriculture productivity. Following factors have contributed for the ineffectiveness of WUA for collective activities.

1. External elements deciding to distribute the community resources to the members of the outside community would cause dysfunctional WUA. When the system is extended to include new members during rehabilitation with out proper consent and consultation, the previous members of WUA would tend to be uncooperative. Hence, WUA would be owned by only one section. This often happens when larger new area is attempted to be included in the rehabilitation of the system in order to reduce the so-called cost of investment per unit of land during assistance to FMIS from the government.

2. When water right issue among the members and of the system is not properly analyzed during rehabilitation, the WUA gets ineffective.

3. When ready made rules and regulations are given to the WUA s from outside forthem to use, they would not match with the social fabric and norms of the society. This will cause the dysfunction of the WUA. Hence, the rules and regulations are to be developed by the concerned WUAs. Each irrigation system is different so rulesand regulations have to match those differences and values of the community. Ignorance of this factor results into ineffective WUAs.

4. Development of trust among the members of the WUA is important. It does not usually allow evolving the trust among the members of the WUA due to time constraint to complete physical target. Membership criteria are important. Resource sharing is going to take place among themselves. If membership is not clear, thentrust among the members erodes.

FMIS AT THE CROSSROAD

FMIS is now at the crossroad. There are many challenges to FMIS. They are the challenges of design, of construction materials due to the depletion of the local construction materials, competition on the use of water, stagnated economic development, new legislation either ignored the existence of FMIS or attempt is made to bring these systems under the control of local administration ignoring the need for the development of polycentric system to strengthen the democratic values at the grassroots level and process of assistance by the government to FMIS.

1. ***Construction and Repair Materials***: These irrigation systems require repair and maintenance regularly. Previously, the repair materials would be used from the forest products. Depletion of forest resources and unavailability of these local materials, the farmers have to depend on imported materials like gabion wire and other construction materials. This condition has made these systems dependent on external resources andgovernment assistance program.

2. ***The Assistance to FMIS:*** The assistance funds to FMIS from loan and donors were channeled through the government. Hence, those autonomously managing systems are brought under the influence of the government. The trend of the dependency has increased resulting into the depletion of the initiative of the local community to manage their natural resources like water and land. Similarly, the depletion of the local construction materials for maintenance of the FMIS, new construction materials like cement, gabion wire replaced the local materials. The government distributes these materials so FMIS's dependency has increased.

3. ***Competitive use of Water:*** Competitive use of water and privatization of small-scale hydropower development have put pressure on FMIS. Water source is the same for irrigation, drinking water and hydropower development. Previously, irrigation alone was monopolizing the use of water but it has changed and put pressure on FMIS. Gradually, share of water in irrigation and agriculture sector is changing.

4. ***Subsistence Economy***: Due to stagnated economic development for long period of time, the return from agriculture has not been significant so the youths of the rural area migrated to urban area and other countries in search of job. The maintenance of FMIS is basically labor-intensive one. The unavailability of youth muscle power in the rural area has impact on the management of FMIS in Nepal. This situation has brought changes in the community control over resource management.

5. ***Introduction of centralized water control system:*** The edit of Ram Saha declared thatthe irrigation management is the responsibility of the community. It also mentioned that the conflict on the use of drinking water is to be settled mutually within the community. Hence, water is considered as "community resource" to be managed and maintained through the collective decision of the community. The extraction, allocation, distribution are to be collectively decided by the community. A number of legal instruments were promulgated with long term impact on the community resource management. "The Water Resource Act,1992", specifies that "Water" is state resources so the uses of water is to be licensed by the government. It gradually moved from community ownership of resource to state ownership concept. Provision is made that the systems which are candidates for rehabilitation from government resources to have the water users associations of the system registered under District Water Resources Committee provided by Water Resource Act, 1992. Such Water Users Associations (WUAs) got legal status and legal recognition. The other FMIS which did not have rehabilitation fund support are considered not legal. Large numbers of systems fall under this category. Through the rehabilitation program, government created two types of FMIS; those government-assisted systems with so called legal WUA and other systems without legal WUAs.

6. ***Newly formed People's Organization:*** The National Federation of Water Users Associations, which has recently formed in Nepal, has serious problem in identifying membership to the federation. At present the so-called legally recognized WUAs are made the members. Hence large number of WUAs of FMIS are kept outside of the bargaining power of the National Federation of Water Users Associations. Only those systems which received the government assistance and those registered in the government agency became the members so the Federation of the Water Users Association represents only small section of officer oriented water users associations (Irrigation Rules and Regulations,1999. The expected activity of the Federation of the Water Users Association in interacting with state agencies, and donors in terms of natural resource management, keeping clear from the political party influences and advocating a seat at the policy and program dialogues will not be possible to be

materialized due to narrow base of its organization. It will not be able to establish itself as people's organization for natural resource management.

7. ***New Legislation:*** As the Water Resources Act, 1992, made water resources of Nepal as state property, the Local Government Act, 1999, made the provision that the local irrigation systems are to be managed by the Village Development Committees of the village. This provision directly interferes with the concept of polycentric society and community resource management at the grassroots. The users groups have onlysuperficial existence under the provisions of these legal systems. The Irrigation Regulation, 1999, states that WUA will be registered in District Irrigation Office of the Department of Irrigation. It is also mentioned that the District Irrigation Office with the approval of the Department of Irrigation can dismiss or suspend the WUA. The new irrigation regulation reinforced the establishment of officer-centered WUAs.Such WUAs would not be conducive for community resource management activity. These WUAs would act only as the extension of the Department.

The trend in Nepal shows that Water Users Associations are moving from community based organization to local government directed institution or government induced WUA organization under management transfer program. Following statement on state of water resource management for irrigation is better described in the following statement.

> "As water resources have fallen under centralized and state control through bureaucracies, policies and legal instruments, communities have had to struggle to maintaining their rights, customary, local practices and livelihood". (Ujjawal,Pradhan , 2000. Page 1 *Water for Life*).

Private sector involvement and contracting
see in next chapter

5. Engineering Costing and Economic Analysis or, Economics of Irrigation Project

Irrigation Service Fee (ISF)

The calculation of ISF for a working irrigated area (IA), with an account for its proper interests, includes the calculation of the tariff rates per 1 ha of irrigated area and 1 m3 of delivered water, as expressed in national currency (n.c.). It is performed from the formulas:

$$ISF = \mathrm{AOC}\frac{(1+\mathrm{P})}{\mathrm{IA}}$$

Where, ISF = Irrigation Service Fee for 1 ha, Rs/ha
AOC = actual annual normal operating costs in NRs
P = Probability (%)
IA = Area Irrigated by the Irrigated system

AOC includes:

- The total management charges of departments, pump stations, electric power stations, and drain wells;
- Maintenance of production buildings, infrastructure (civil buildings, communication facilities, and roads), and transportation facilities;
- Care of plantations;
- Equipment costs;
- Depreciation charges;
- Interest expenses on short-term credits, and
- Insurance contributions; etc.

Cubic meter tariff rate:

$$\mathrm{ISF} = \frac{\mathrm{Twf.Vwi} + \mathrm{OEv}(1+\mathrm{P})}{\mathrm{Vwd}}$$

where

ISF is the tariff for 1 m3 of water (in n.c.);
Twf is the price of 1 m3 water taken from the WF (n.c.);
Vwi is the total volume of water intake from the water facility, m3 ;
Vwd is the total volume of water delivery for irrigation, m3 .
OEv (in n.c.) denotes conventionally variable operating costs, including:

- maintenance of hydraulic structures, gauging stations, barrages, channels, pumping stations, electric power stations, and wells;
- electricity charges;
- cleaning of the irrigation system;
- protecting, regulating, and flood-control works;
- leveling of dams and channels, and
- provision of emergency stores.

Benefits of irrigation

- Contribution of irrigation to agricultural productivity
- Food supply expansion
  - Irrigation and agricultural land expansion
  - Irrigation and increased crop yields
  - Irrigation and double cropping of land
- Welfare improvements
  - Irrigation, employment opportunities and income
  - Irrigation and land values
- Irrigation supply stabilization
- Environmental benefits
- Benefits of the conjunctive use of groundwater and surface water
- Benefits of flood control

Cost of irrigation

- Capital Costs
- Environmental Costs
- Dynamic costs of water resources
- Social concerns
- Overuse of groundwater resources

Cost Benefit Analysis
Sensitivity Analysis

Economic Analysis of Irrigation Projects

Decision Criteria

- Net Present Value

Net Present Value (NPV) is defined as the present value of net incremental benefit, namely total benefit minus total cost at present value.
Following the formula to calculate NPV:

- Cost-Benefit Ratio

Benefit Cost Ratio (B/C Ratio) is the ratio obtained when the present value of benefit is divided by the present value of cost. The B/C ratio is more than 1, the project is economically feasible.
The formula is as follows:

- Internal Rate of Return

Prior to the calculation of NPV and B/C Ratio, it is necessary to decide the discount rate in calculating these criteria and the results of calculation differ by applied discount rate. For the case above, 10% of discount rate was applied to calculate NPV and B/C Ratio but for example, if 5% and 15% of discount rates are applied for the same cash flow sheet.

Monitoring and Evaluation, and feed back in Irrigation Sector

Monitoring and evaluation of irrigation projects is a neglected subject in the past. Without continuing and effective monitoring and evaluation system, it is unlikely that the project benefits can be achieved or they will be optimal.

Irrigation projects have thus generated both extreme optimism and pessimism, especially in recent years. Undoubtedly a major reason for the existence of such diametrically opposite views is due to the lack of effective monitoring and evaluation (M and E) of irrigation projects. M and E process has received much lip service during the past decade, but has seldom been carried out comprehensively on a continuing basis. The analysis of US AID's experiences in irrigation projects indicated that Mand E activities of both donor and recipient countries have "come in for criticism from each group about its own organization and about the activities of its counterpart," and that "too little of it gets done by either group" (Steinberg, 1983).

Monitoring is defined as continuous or periodic surveillance over the implementation of an activity (and its various components) to ensure that input deliveries, work schedules, targeted outputs and other required action are proceeding according to the plan. Since the purpose of monitoring is to achieve efficient and effective project performance, it is an integral part of the management information system and is an internal activity.

Evaluation is defined as "a process which attempts to determine as systematically and objectively as possible the relevance, effectiveness and impact of activities in the light of their objectives. It is learning and action-oriented management tool and an organization process for improving activities still in progress and future and planning, programming and decision-making."

It can be persuasively argued that because of the indifferent past performances of irrigation projects in achieving their stipulated objectives, it is absolutely essential that monitoring and evaluation become an integral part of the management process to ensure future stream of benefits occur to the right target group. It can be equally argued that one of the main reasons for the failure of irrigation projects to meet the approved objectives in the past is due to the lack of appropriate monitoring and evaluation, and the failure by the management to use monitoring and evaluation successfully as a management tool. Figure 1 outlines the operational aspects of the M and E system. Monitoring and evaluation for irrigation projects have many requirements, the principal ones for most purposes are the following:

- (i) timeliness;
- (ii) cost-effectiveness;
- (iii) maximum coverage;
- (iv) minimum measurement error;
- (v) minimum sampling error; and
- (vi) bias-free.

There is also a tendency in irrigation projects which are multi-faceted, to introduce biases in terms of one's own discipline. Thus, M and E carried out byundisciplinary people often tend to emphasize areas that are of primary interest to them. The problem emphasis of different disciplines in the area of irrigation, and the standard solutions proposed could be the following

Discipline	Problem emphasis	Solution
Aministrator	Poor Coordination	New organization with administrator as coordinator
Agricultural economics	Agricultural prices and marketing, Lack of credit, Risks of production	Improve marketing and prices, provide credit, reduce risks
Agricultural extensionists	Farmers unaware of good agricultural and water management practices	More extension services to farmers
Biologist	Inundation impacts on flora	Reduce impacts by changing

	and fauna	scale or location
Economists	Inefficient water use, low return on capital and underutilization of potential	Water pricing, more investment
Engineers Agricultural	Poor land levelling, poor maintenance or lack of field channels	Level land, improve situation
Engineers Civil	Inadequate structural development, poor operation and maintenance (O&M)	Construct more/better structures, provide more funds for O&M
Engineers Drainage	Salinity and waterlogging	Construct comprehensive drainage system
Environmentalists	Too much damage to the environment and ecosystems	Stop construction or reduce scale of development
Lawyers	Central-Provisional relations or international implications	Resolve potential legal problems
Political Scientist	Inequitable distribution of agricultural production and water	Change power structure

Construction Management

Construction Technology

Construction technology is that branch of engineering which deals with all kinds of activities and technology or operations for changing existing ground in the designed shape, slope, and to provide all necessary facilities for smooth and efficient operation of irrigation canal and also include the reconstruction of existing Canal. As per the nature and type of works and elements of irrigation project to be constructed various activities can broadly divided into several works.

1. Site clearance and Earthwork
    - Site clearance
    - Earthwork for cutting and filling
    - Excavation for borrow pit
    - Excavation for structural foundation
    - Disposal of surplus earth
2. Drainage works
    - Side drains
    - Causeway
    - Vented or Flood bridge
    - Culverts
    - VRB
    - Minor bridge
    - Major bridge
3. Protection works
    - Earth retaining structures
    - River training works
    - Gully control works
    - Land slide stabilization
    - Bridge protection works
4. Pavement works
    - Sub grade works
    - Sub base works
    - Base works
    - Surface works
5. Miscellaneous works
    - Road ancillaries
    - Traffic sign/markings etc
    - Road furniture

- canal Furniture
- Bio engineering works
- Public awareness about road and traffic

Construction Tools, Equipment, and Plants

Although the road construction may be done manually but it takes lot of time to complete the irrigation project. The quality of the works may not be achieved to the desired degree and which cannot maintained strictly by using intensive labour force within time in comparison to construction equipment. In developing countries like Nepal the trend of using construction equipment increases rapidly. Construction equipment used in road construction project is:

1. Tools
- Hand shovel
- Chisel
- Peak
- Spade
- Hand rammer
- Brushes
- Trowel
- Wheel barrows etc

2. Equipment

a. Earth moving equipment
- Dozer
- Scraper
- Loader
- Excavator
- Dragline
- Clamshell
- Trench digger

b. Compaction equipment
- Smooth wheel rollers
- Vibrating rollers
- Pneumatic rollers
- Sheep foot rollers
- Rammers

c. Leveling equipment
- Grader

d. Paving equipment
- Binder sprayer
- Aggregate spreader
- Cement concrete mixer
- Bituminous paver
- Cement concrete paver etc
- Air blowers
- Cleaning devices

e. Lifting Equipment
- Backhoe
- Crane

f. Transporting equipment
- Dumping trucks
- Tippers
- Trucks flat body
- Mini dumper
- Tractors

3. Plant
- Cement concrete plant
- Aggregate crusher plant
- Screening plant
- Washing plant
- Sand blowing plant
- Fully maintained laboratory and testing equipment.

CONSTRUCTION TECHNOLOGY:

Based on priority, there are various types and standards of roads. The selection of base/subbase course and the surface course depends upon the following factors:

- Type and Size of Canal
- funds available for construction and maintenance,
- soil and drainage conditions,

- Availability of construction materials at site,
- Climatic Conditions,
- Availability of plants and equipment's,
- Time available for completing the con project,
- Altitude at which construction has to be made,

Quality related all

It is often voiced that quality control is not up to the mark in our constructions. Explain the provisions made for quality assurance in the standard specification and conditions of contracts. Discuss why quality work is not resulting and suggest remedial measures for it.

"गुणस्तरीय पूर्वाधार: सम्बृद्धिको आधार"

Definition

Degree of goodness of fit for purpose is called quality. In quality control, operational inspection and test activities are done at different stages that used to confirm quality requirement has been met or not. Quality is pertinent issue in our construction industries specially in infrastructure project because of one third of annual budget has been allocated for that.

Nepal has taken fill fledge membership of WTO which are for stringent quality control in order to compete in global market. Also, direct and very strong relationship exists between sustainability and quality. Public faith will be lost due to poor quality and also safety issue is related to quality. Hence, quality work should be up to a mark level. Quality control in construction typically involves insuring compliance with minimum standards of material and workmanship in order to insure the performance of the facility according to the design. There minimum standard is contained in the specification.

समस्याको पहिचान: " गुणस्तरीय पूर्वाधारको लागि स्थापित राज्य संयन्त्र असफल हुदै गएको"

Legal provision and policies related to quality

- Standard specification for road and bridges works 2078
- Provision and conditions for WTO membership
- Public procurement Act and Regulation
- Governance Act, 2064
- Professional code of conduct and ethics
- Engineering council act and regulation
- National code of conduct, 2075
- Departmental different directives, guideline, standard design, drawing, methodology for various works.
- Minitrial/Departmental Level Monitoring and Evaluation Branches
- NPC, Constitutional committee etc.
- Various oversight agencies like CIAA, NVC, OPMC, Office of Auditor General etc.

The provisions made for quality assurance in the standard specification and conditions of contracts

1. Departmental strategy

- Strengthening institutional organization of DoR
- Establishing working procedure
- Improving directives, guideline and etc.
- Continuing administrative and technical backup
- Strengthening material testing lab
- Institutionalizing monitoring and evaluation mechanism and auditing
- Strengthening private sector organization in quality work.

2. Quality assurance in the standard specification

3. Quality assurance in the conditions of contracts

Issues of quality

- Man related: availability of professional/skilled and ethical manpower.
- Material related: duplicate/availability of substandard construction materials.
- Money related: concept of "make money any how"
- Method related: neglecting standard procedure and methodology.
- Minute related problems: no enough time for survey, design, construction, supervision and documentation.
- Other issues such as WTO membership and quality, community participation and quality, quality control in small and maintenance works, corruption and quality etc.

Problems (Reasons for not resulting Quality or Finding regarding Quality in DoR)

1. Institutional

- The laboratory and testing facility are inadequate and poorly maintained
- DoR work's concentration is off SRN/NH
- No appropriate mechanism and system of quality monitoring
- Ritual adherence for submission or compliance with QAP, work schedule
- Dual role as site engineer and lab in charge
- Lack of high moral in field staff.

- Lack of appropriate capacity for public procurement and contract administration.

2. Technical

- Absence of professional and trained manpower
- Insufficient staff for lab work
- No proper and well set up in lab like continuing supply of electricity, water supply, maintain temperature etc.
- Test reports are as ritual and only for billing/payment purpose.
- Lack of coordination with IOE/Department of measurement and quality for calibration in time.
- No clear-cut responsibility/accountability between the lab and construction and supervision staffs.
- Hesitation for use of new method and technology for lab and construction works.

3. Financial

- Test reports are as ritual and only for billing/payment purpose.
- No body have priority for well lab set up and operation.
- Insufficient budget allocation by MoF for lab/ considering as general office but in lab there is must be continuing supply of electricity, water supply, maintain temperature etc.
- No additional allowance for lab staffs
- No clearcut guideline/directives for professional liability insurance as per PPA/PPR
- Lack of management of site office/camp (all technical staffs are near offices)
- No any tie up the insurance policy with quality of works.

4. Other

हचुवाको भरमा गरिएको योजना छनौट, हतारमा गरिएकोDPR,राम्रो निर्माण सामग्रीको अभाव, व्यबस्थित प्रयोगशालाको अभाव, काम शुरु गर्ने बेला देखि नै संगठित रुपमा आउने अवरोध जस्तै खानी जन्य पदार्थको निकाशी र उपलब्धता, चन्दा आतंक, दण्डहिनता, स्थानीयबाट बिकासको काममा बिरोध गर्ने वा दलीयराजनीतिक गर्ने परिपाटी, निजी क्षेत्रमा व्यवसायिकताको अभाव आदि।

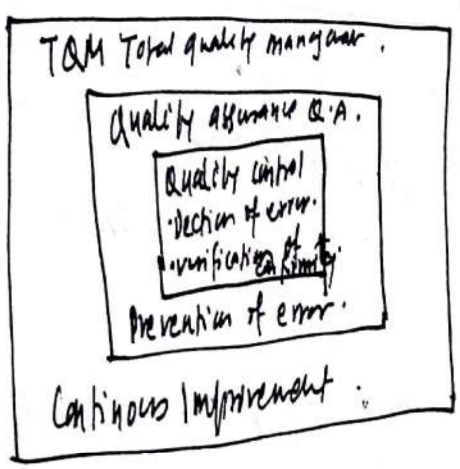


Fig: Quality management.

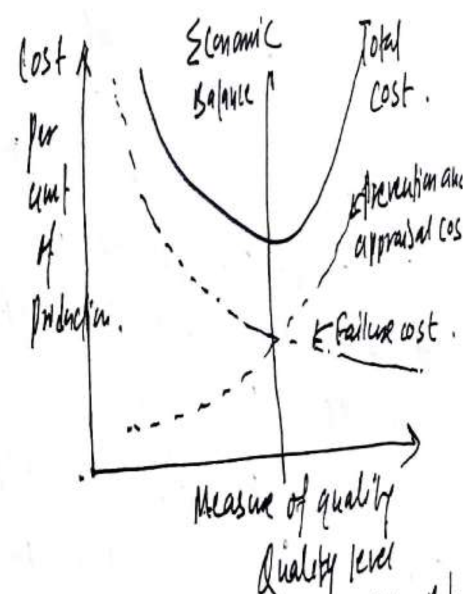


Fig:- Economy of Quality Bala.

Suggestion for Quality Work/ Remedial Measures for Quality

"Let's no work for petty gains and let's work for national interest as a whole"

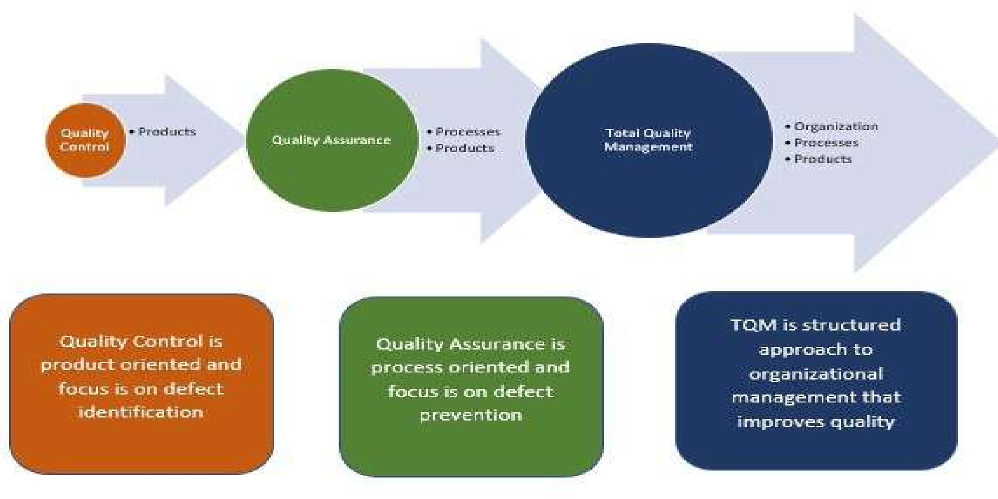
- Improve and rectify the above stated issues/problems efficiently.
- DRO labs to be upgraded with well equipped.

- CRL to be strengthened and fully staffed and establish at least 5 labs as CRL in appropriate location of the country under DoR.
- Appropriate training to the staffs about modern construction method and technology.
- Submission and compliance of QAP/work schedule mandatory.
- Proper work division and site camp/site office should be managed efficiently.
- Strictly follow the PPA/PPR/ design guideline/standard specification/norms etc.
- Update/review and prepare the appropriate guideline/technical note as per the work nature.
- Use of appropriate technology such as bio engineering/MRE approach for gully erosion/stability of slope etc.
- Application of quality and performance audit in all major part/works.
- Tie up the work performance to the reward and punishment of staffs.
- Strengthened the over sight agency like NPC/NVC with appropriate norms and guidelines.
- Clear guideline and directives for PLI
- Always be careful by all concern party and staff from the selection of the project to the operation and maintenance with respect to the sustainability and quality.
- Always careful about the quality material, skilled manpower, appropriate methodology, and modern technology for best quality.
- Promotion of modern technology such as patch master, velocity machine for patch, recycling technology for pavement overlay, x ray machine, bridge inspection vehicles, TBM for tunnel, infra doctor for pavement evaluation, RMC for concreting, trench less technology, drone supervision, precast technology, pile boring machine, failure bencher for tree cutting, Oster berg cell or Dynamic test kentledge for test of pile as per the site condition.
- Proper coordination of stakeholder and agency
- Effective monitoring and evaluation of the work and workmanship including concerns all staffs.
- Compliancethe accountability and moral along with the professional code of ethics.

निष्कर्ष

गुणस्तरीय पूर्वाधार मार्फत दिगो बिकासको लक्ष्य सहित समृद्ध नेपाल र सुखी नेपाली हुने चाहनालाई सकार पार्न हाम्रो कार्य संस्कृतिमा रहेको परम्परागत सोंच, नीति, नियम, प्रक्रिया, मापदण्ड आदिमा "Paradigmshift" गर्दै समय सापेक्ष दक्ष जनशक्ति, गुणस्तरीय निर्माण सामग्रीको उत्पादन/वितरण सहित modern technology, working procedure /standard/norms आदिको प्रयोग गर्ने संस्कृतिलाई सबैले आत्मसात गर्नुपर्ने देखिन्छ ।

TQM, Quality Control and Quality Assurance



Distinguish between QC, QA and TQM

Criteria	Total Quality Management	Quality Contro
Software difference	TQM focuses on continuous improvement in the processes for making the software	QC is concerne that a product n prescribed tech of quality and m customer's requ
Types of experiments performed	Causal research that analyses the effect of the independent variable on the dependent variable. It helps find the effectiveness of TQM implementation	Experiments rel inspection, revi where to set the the output as de
Design of experiments	DOE that focuses on continuous improvements can be used here. e.g. Taguchi Method is a process/product optimization method that is based on 8-steps of planning, conducting, and evaluating results of matrix experiments to determine the best levels of control factors. The primary goal is to keep the variance in the output very low even in the presence of noise input	DOE that focus can be used he deal with planni analyzing, and i controlled tests factors that con a parameter or parameters. Th related to quant e.g. Causal-Compar Experimental. It establish cause relationships an variables
Statistical techniques	Statistical methods for process capability analysis are used here. Assessment of capabilities of a process/ machine relating to expectations of a client is widely understood (a client can be the subsequent process) In this range, it determines the capability indices of a process/machine. Focus is to reduce variations	The 7 Quality C Tools are used - Cause-and-eff (also called Ishi or -fishbone dia - Check sheet - Control chart - Histogram - Pareto chart - Scatter diagra - Stratification
Department metrics	The Baldrige framework was used to identify six key measures of TQM success: - Management involvement - Strategic quality planning - Employee involvement - Training - Process capability - Customer perceptions	Metrics are bas in defects, Defe Defect density i
Performance goals	Goals are measured based on TQM concepts, namely Customer Focus, Leadership, Teamwork, Continuous Improvement, Measurement and Benchmarking, QFD (Quality Function Deployment) to translate Customer Needs into metrics	Current defects and defect dens Performance go to reduce these percentage in a
Certification	Total Quality Management Professional (TQMP) Lean Six-Sigma Certifications	Quality Control Quality Assuran
Eligibility	There are no pre-requisites	There are no pr
	The program covers all basic TQM principles and gives the	They instruct st control techniq

QUALITY CONTROL: Verifying the Quality of the Output.

- **This is the process by which entities review the quality of all factors involved in the production. This covers cycles from receiving materials and manufacturing to testing, packing and shipping. So, it is product oriented and focuses on detect identification.**
- **Quality control is the most basic level of quality management. It includes all activities of inspecting, testing, or checking a product to ensure it meets the requirements.**
- **The intent of QC is to identify any issues—and either fix them or eliminate them—to make sure the end result is as expected. QC is typically conducted reactively, at the end of the process**

QUALITY ASSURANCE: Managing and Planning for Quality

- **Quality assurance takes your quality management process a step further. QA is focused on planning, documenting, and agreeing on the steps, rules, and guidelines that are necessary to ensuring quality. The planning happens at the beginning of a project, and the end result is a documented quality plan.**
- **The main purpose of QA is to prevent defects from entering into your product in the first place, so it's a proactive measure to ensure quality. Planning for quality is key to mitigating risks, but also saves you a lot of time and money.**
- **Contractor implements quality control in compliance wit approved QAP. Engineers' approval of QAP not relieve contractor from his responsibility works to be performed as per specification.**
- **Engineer's approval of QAP not exempt the contractor of any procedure to inform the engineer in writing a request for engineers' approval.**
- **Contractor shall monitor and update QAP as per the requirement and instruction of Engineer.**

Strategy of Quality assurance in Nepal.

- **Introduce the rule of "do it right at first time".**
- **Introduce the rule to get quality at first time.**
- **Quality audit to enforce to performance the all-sequential activities of quality control system.**
- **Right at design stage, the quality assurance system needs to be considered since choice of material and workmanship lead quality achievement their availability.**
- **Defect liability period should be set 5 years.**
- **QAP should be based on detailed project programme of work.**

Policy of QAP or policy for quality work in sequential order

- **Use of quality material.**
- **Use of skilled and experienced manpower.**
- **Use of standard equipment and tools.**
- **Use of standard methodology for workers.**
- **Product will be within the specified quality and requirement.**
- **Proper monitoring and evaluation.**

QAP includes

- **The sequential of work step by step**
- **Quality control schedule ***
- **List of sources of material and manufactured certificate, their main characteristics etc.**
- **Testing, team, lab and site test plan**
- **List of tests and quality control procedures to be implemented by sub-contractor if any**
- **Organization for quality control and procedure and their man power.**
- **System used to M and E the aspect of project, services facility to determine of quality standard are being met.**

Quality control schedule *

- **Summary of test schedule and testing programme detailing list of tests for compliance, laboratory trials, construction control tests their frequencies tests for acceptance of completed works.**
- **Summary of list of critical acceptance testing procedure corresponding to tasks on critical path according to construction programme.**
- **Estimate of number of tests to be carried out at outside of lab and mention name of lab proposed to carry out tests.**

TOTAL QUALITY MANAGEMENT

- **It is a management approach and built with three different things like method, purpose and system. Firstly, system includes all persons of all divisions at every level, secondly, the method runs itself with the management method and analytical method. Thirdly, purpose absorbs the quality, cost, environment, delivery and safety.**
- **The important 6 C's of TQM are commitment, culture, continuous improvement, cooperation, customer focus, and control.**
- **TQM is the integration of all functions and processes within an organization in order to achieve continuous improvement of the quality of goods and services. Integrated organizational effort designed to improve quality of processes at every business level.TQM may also be defined as performance superiority in delighting customers.**

Total quality management (TQM) means:

1. Satisfying customers first time, every time;

2. Enabling the employees to solve problems and eliminate wastage;

3. A style of working, a culture more than a management technique;

4. Philosophy of continuous improvement, never ending, only achievable by/or through people.

Quality management system (QMS) and Quality Management Plan (QMP)

QMS ensure that the intended degree of excellence is attained.

QMS have three elements

1. QAP
2. Quality control Process (QCP)
3. Quality Audit system (QAS): tracking and documentation of quality assurance and quality control program.

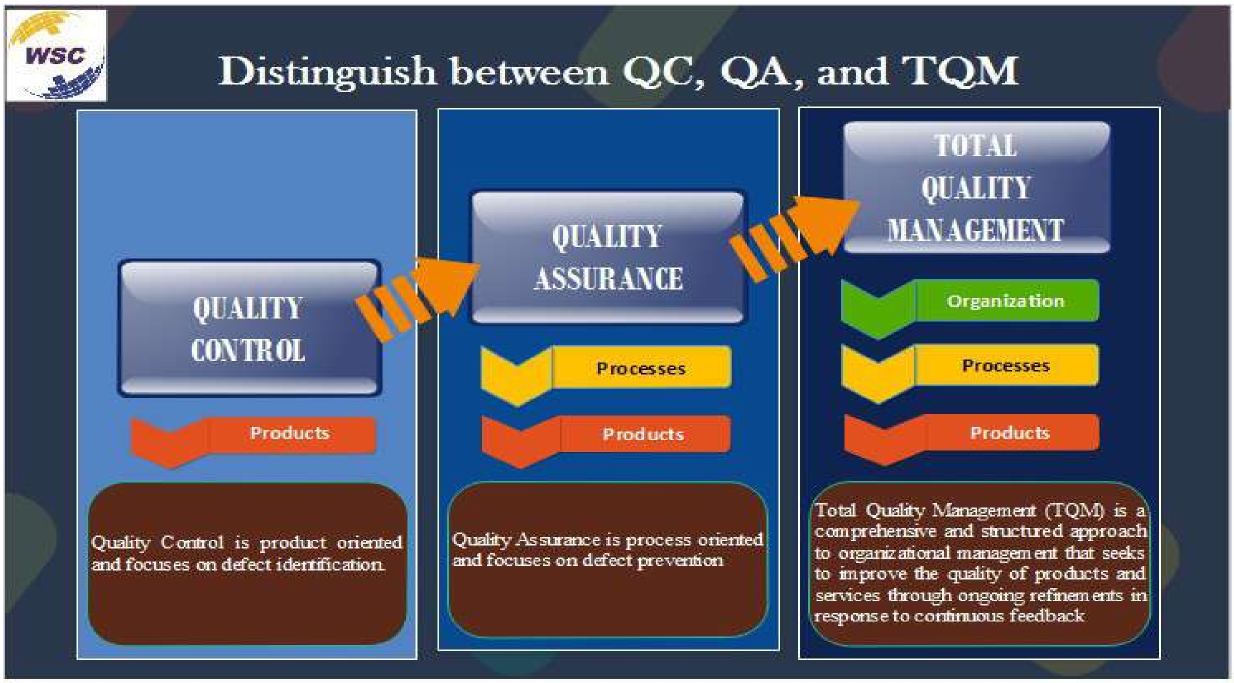
Quality management plan

Define the acceptable level of quality which is typically defined by the client and describes how the project will ensure this level of quality in its deliverables and work progress. To get quality, steps are

- Quality design
- Quality assurance plan approval and implementation
- Following the quality policy (as stated in above)
- Strong implementation of quality control system.

Elements of quality assurance system for road projects

1. Assessment of requirement of road project: design criteria/ design life minimum acceptable level of riding quality, higher the standard of road, higher will be the cost.
2. Choice of quality materials and design: evaluating pavement thickness and composition for the assessed traffic and sub grade conditions to meet the design and riding quality.
3. Development of technical specification and acceptance criteria.
4. Choice of construction method, equipment and plant must meet requirement of technical specification as demanded by design criteria.
5. Field supervision and quality control of material, construction, techniques, surface finish to desirable profiles such as
    - Inspection and testing of material, production process and the end product
    - Measuring variations from the predetermined standards
    - Taking corrective action to minimize adverse variation and
    - Accepting and rejecting the works.
6. Assessment of quality finished road.
7. Periodic inspection and maintenance measures during DLP and after that also in periodic interval as per the pavement evaluation.


WSC
Distinguish between QC, QA, and TQM
QUALITY CONTROL
Products
Quality Control is product oriented and focuses on defect identification.
QUALITY ASSURANCE
Processes
Products
Quality Assurance is process oriented and focuses on defect prevention
TOTAL QUALITY MANAGEMENT
Organization
Processes
Products
Total Quality Management (TQM) is a comprehensive and structured approach to organizational management that seeks to improve the quality of products and services through ongoing refinements in response to continuous feedback

Credit: diffen.com	Quality Assurance	Quality C
Definition	QA is a set of activities for ensuring **quality in the processes** by which products are developed.	QC is a set of activit **quality in products,** identifying defects i produced.
Focus	QA is a **proactive quality process** which aims to prevent defects in the process used to make the product.	QC is a **reactive pr** identify (and corre the finished produ
Goal	To **improve development and test processes** to reduce defects when the product is being developed.	To **identify defects** product before it's
How	QA establishes good quality management systems and the assessment of its adequacy and conformance audits of the system.	QC find & eliminat quality problems t equipment so that requirements are
What	Prevention of quality problems through planned and systematic activities including documentation.	The activities or te to achieve and ma product quality, pr service.
Responsibility	Everyone on the team involved in developing the product is responsible for quality assurance.	Quality control is u the responsibility team that tests th defects.
Example	Verification is an example of QA	Validation/Softwa example of QC
Techniques	Statistical Tools & Techniques can be applied in both QA & QC. When they	When statistical to techniques are ap

Construction management (QM)

- The management of works starts much before the actual commencement of work. It includes activities such as invitation of tenders, selection of contractors, mobilization and actual execution.
- The following aspect deserve careful consideration in construction management.

1. Management of materials.
2. Management of labor.
3. Management of equipment.
4. Management of finance, fund etc.

- Efficient management of materials includes activities such as assessment of requirement, location of sources and supply and purchase, transport, storage and issue on works.

- The procurement should be so phased that works do not suffer at any stage due to lack of material and at the same time, the stock of material is not unnecessarily high.
- Ensure about the adequate supply of labor, amenities to labor such as temporary housing, medical facilities.
- Equipment management is one of prime importance because of high cost. The efficient equipment management should be done to keep in view.
    - Selecting of proper size, number and specified equipment to do the work.
    - Preparation of utilization programme.
    - Experienced operators and adequate maintenance.
    - Safety aspects.
- Financial management covers budgeting, keeping proper account ensuring adequate flow of funds and keeping watch over the financial progress.
- Construction Management plan (QMP) cover
    - Construction methodology for each item with cross reference to specification.
    - Construction schedule based on CPM and PERT method.
    - Cash flow pattern with respect to construction schedule and methodology presented in the form of S curve.
    - Quality assurance plan (as stated in above)
    - Project monitoring system: design of appropriate monitoring system, monitor construction methodology, construction schedule, cash flow pattern etc.

5. Maintenance and Rehabilitation

Arbitration Act and Dispute related issue

Contract Delays

Introduction

Contract have completion dates. Bonus and liquidated damages are tied to completion date. Delay by the employer can frustrate liquidated damages clauses whereas, delay by the contractor can result in termination of contract and also leads on to financial matters

In general delay in construction are of three types, they are:

Excusable Delay

A delay that entitles the contractor to additional time for completion of the contract work, generally arising from causes beyond the contractor's control is excusable delay. Excusable delays may be classified further as excusable compensatory delays and excusable non-compensatory delays. Whether delay is classified as compensatory or no compensatory depends primarily on the terms of the contract.

An excusable delay can occur due to various factors, which can be classified into two categories:

- Beyond the control or without the fault of either party (Excusable non-compensatory).
- Within the owner's or his representative's control (Excusable compensatory)

In the first case, the contractor is entitled to get extension of contract performance time while the later will allow the contractor both time extension and additional cost.

When delays are excusable, the contractor will not be subject to liquidated damages, nor can the contractor be terminated for default due to such delays. Liquidated damages constitute the specified amount that a contractor will to the owner for non-excused late completion. Whether the contractor can recover the delay cost for an excusable delay depends on whether the delay is compensatory or non-compensatory or whether it is concurrent with other delays. Examples of excusable delays caused by different factors are:

Delay Caused by Owner

- Failure to provide a project site
- Late notice to proceed
- Failure to provide proper financing
- Failure to provide owner's furnished materials or components
- Interfering with or obstructing work on the project

Delay Caused by Architect/Engineer

- Defective plans and specifications
- Failure to provide drawings on schedule
- Delay in review or approval of shop drawings
- Delay in change orders
- Stop-work order

Delay not caused by Any Party or Participant

- Acts of God
- Act of public enemy

- Unusual delays in transportation, such as a freight embargo
- Epidemics
- Unusual weather conditions (force majure)
- Strikes

Excusable Compensatory Delay
A delay that entitles the contractor to extended field office costs and perhaps home office costs, as well as additional project time is excusable compensatory delay.

Excusable compensatory delays are due to acts or omissions of the owner or owner's representatives. This type of delay entitles a contractor to additional compensation for costs of delays and time of project completion. A delay can be compensable solely by causing damages for the contractor. Typically, excusable compensatory delays are attributable to change order or to owner's actions that change the contracted requests.

In the contract, the compensation provision may allow extension of time or compensation for additional costs, but frequently the extension of time is the sole remedy for delays. In this case, if the contractor seeks compensation, they have to file a lawsuit for delay damage cost. Examples of excusable compensatory delay are:

Delay caused by Owner

- Failure to provide a project site
- Late notice to proceed
- Failure to provide proper financing
- Failure to provide owner furnished materials or components
- Interfering with or obstructing work on the project

Delay Caused by Architect/Engineer

- Defective plans and specifications
- Failure to provide drawings on schedule
- Delay in review or approval of shop drawings
- Delay in change orders
- Stop-work order

Excusable Non- compensatory Delay

A delay that entitles the contractor to additional time for completion of the contract work but no additional compensation is excusable non-compensatory delay. Excusable non- compensatory delays are not caused by the owner, designer, contractor, subcontractors, suppliers, or other parties in the design and construction process. Because this delay is beyond the control of any of the parties, contract and case laws generally minimize the risk to all parties by a compromise:
'The contractor's late completion will be allowed equal to the amount of delay, but no additional compensation will be awarded. Most contracts contain written statements that deal specifically with this type of delay. Examples of non-compensatory delay are:

Delay not caused by Any Party or Participant

- Acts of God
- Act of public enemy
- Unusual delays in transportation, such as a freight embargo
- Epidemics
- Unusual weather conditions (force majeure)
- Strikes

Non-excusable Delay
A delay that does not entitle the contractor to either additional time for completion of the contract works or additional compensation is non-excusable delay. Such a delay may be non-excusable due to the contractor's failure to meet its contractual obligations or due to the terms of the contract.

A non-excusable delay is within the contractor's control and could have been avoided. This type of delay does not allow the contractor to recover any additional time or cost. Conversely, such delay could be compensable to the owner in the form of liquidated or actual damages paid by the contractor for late completion or increased cost to accelerate the work. Furthermore, the non-excusable delay may constitute a breach of the construction contract by the contractor and may justify the termination of the construction contract.

The owner normally is in a difficult position to identify the non-excusable delays at the early stages because he seldom maintains the construction schedule with sufficient detail to pinpoint the contractor's delay. This type of delay, therefore, is identified when the dispute arises. A contractor, on the other hand, is more likely to maintain the detailed schedule, so he is in a better position to monitor job progress and identify delays, which are attributable to the owner. Examples of non-excusable delay are:

Delay caused by Contractor

- Slow mobilization
- Inadequate labor force

- Strike caused by unfair labor practice
- Poor workmanship
- Late delivery of materials and components
- Failure to coordinate multiple sub contractors

Concurrent Delay
The concurrence of two or more delays arising from independent causes and affecting a project during the same or overlapping time periods is concurrent delay. Concurrent delays may act jointly to affect a single activity or path, or may act independently to affect multiple activities or paths.

Concurrent delays are two or more delays that occur at least to some degree simultaneously. As used in construction law, the term refers to the situation when there is more than one delay occurring at the same time, each of which, if it had occurred alone, would have affected the project completion date.

Courts determine the legal impact of concurrent delays by examining the responsibility for the concurrent delays and determining whether the parties are seeking compensation or an extension to time. The concurrent delay can be more than one type of delay. With respect to contractor recovery for concurrent delays, the delays must be solely the owner's responsibility. Similarly, if the owner can clearly distinguish the contractor's responsibility for concurrent delays, the owner can collect liquidated damages. In general, when excusable and non-excusable delays are concurrent, the contractor ought to be entitled an extension of construction time. In case of concurrent compensatory and non-compensatory delays, the contractor should be entitled to a time extension but not to damages. For the contractor to collect damages, the owner would have to cause all compensatory delays.

Excusable + Non-Excusable ⇒ Time extension
Concurrent (Compensatory + Non-compensatory) ⇒ Time extension only
All compensatory delay solely by owner ⇒ Compensation + Time extension

If the concurrent delays consist of delays attributed to both the owner and contractor, some cases hold that neither can recover damages for the other's act. Some endeavor should be made to apportion the concurrent delays between parties. Inadequate documentation may, however, make apportionment impossible. If concurrent delays cannot be apportioned, neither the owner nor the contractor can recover delay damages.

Extension of time
Time is essence of the contract. It is necessary to obtain program from the contractor in the form specified in the contract and revised program from time to time. This will ease the process to:

- assess and compare program achieved against original program
- to obtain monthly or biweekly program
- to monitor the actual progress of the work
- to check that escalation is not claimed for work done later
- to take actions and resolve issues in time
- to assess the effect of variations on completion of the project

Major causes that have implication on time

- variations that involve time and additional cost
- design change
- scope of work is changed
- time lag between project preparation and implementation
- unrealistic estimate
- incompetent contractors
- improper plant and equipment
- litigation
- slack in supervision
- slow decision making
- absence of fund flow
- delay in land acquisition
- local law and problems
- delay in shifting of utilities

Provision of extension of time
Extension of time effect
Extension of time due to employer's default: time extension is awarded to the contractor without imposing liquidated damages. Employer to bear all additional costs under the contract price escalation to be borne by the employer.
Extension of time due to contractor's default:
Time extension is awarded to the contractor by imposing liquidated damages. Extension of time in such cases may have implication on other aspects of the contract such as price escalation. Contractor to bear additional costs at his own expense.

Liquidated damages
Liquidated damages are designed to reflect what the owner reasonably estimates the economic damages will be in the event of late delivery or completion.
Contractors with liquidated damages clauses should also contain excusable clauses like strikes, weather, natural disasters etc.
Liquidated damages are remedies available to any contracting party to compensate for the financial loss suffered as the result of a proven breach of contract.
Liquidated damage acts as a deterrent to the contractor not to take the contract works casually.
In common law countries, it is known as liquidated damages in civil law countries, it is known as contractual penalties
Provisions of liquidated damages

Contract Variations

Introduction
Variation literally means change, alteration, modification etc. In general Variation Order (VO) is categorized as two types:
- Change in original Bill of Quantities (BoQ)
- Additional work items or Specifications changes

Contracts provide clauses for execution of varied works. Variation clause is a tool which allows Project Manager/Employer to modify the contract as required during the performance of contract. It allows to execute unforeseen works without breach of contract or a new contract.

According to Institution of civil Engineers (ICE): "Variation are required for the satisfactory completion and functioning of works". Greater the elements of unknown more chances of variations. Strictly speaking, contractor is not bound to execute more than contracted unless contractual provisions bind him. The binding documents are:
- Contract documents for the Contractor and
- Contract documents, financial rules, procurement act and rules, donor's guidelines for Employer's representative.

Causes of variations
Variation occurs with the change in circumstances in different stages of the project. The various causes that have direct impact on variation are:
- Changes in quantities without change in scope/design
- Change in scope of work: Compromise/design improvement and additional work.
  Mainly due to shortage/availability of additional fund
- Technical reasons: These covers
  - Changes in design/specifications, which occurs due to lack of information, raising/reducing design parameters and additional requirement due to further investigation/information and
  - Inadequate/faulty design
- Change in site condition: This may arise due to natural phenomenon, further deterioration of existing condition and due to time lag between design and execution
- Time extension: General items need to be varied in this process
- Change in specified sequence or timing
- Time constraint for new contract

Effect of variation
Variation has a important effect in contract execution, performance and management. The various effects are;
- Satisfactory completion and functioning of works
- Reduction or increment in scope of work
- Reduction or increase in volume of work
- Increment or reduction in project cost
- Time extension
- Increment in price escalation cost
- Revision of contract rates
- Fixation of new rates
- Claim situation

Variation related clauses
Public Procurement Act 2063 clause 54 and Public Procurement Regulations 2064 clause 118 provides variations limited up to 15 % to be approved by the department chief, up to 25% to be approved by Secretary of Ministry and in excess of that to be forwarded and approved by the cabinet.

Variation preparation and approval
Flow charts are prepared if needed followed by Standard formats and submitted to the Approval authority. Additional requirement of VO committee may be needed for adequate justifications with supporting documents, submission after approval within authority. New rate negotiation is done if needed as well.

Benefits and risks associated with variation

Benefits are:

- Financial management
- Satisfactory completion and functioning of works as planned
- Scope and technical management
- Facilitates inclusion of changes required during construction
- No contract variation may result in: incomplete work, desired quality may not be achieved, claims and loss to the employer

Risks are:

- Increase in project cost: due to additional works, extension of contract period, additional escalation cost due to time extension, additional supervision cost and claim from contractor
- Delay in project completion
- Absence of additional fund leads to scope curtailment
- Probable high work item rate pricing for varied works due to uncompetitive rate arrived through negotiation
- Rate negotiation generally an advantage to the contractor
- Competitive bid could result to be more costly due to increase in BoQ item quantity with escalated rates
- Opportunity for non-performing contractor for excuse for time extension
- Contract litigation

Conclusion

Although variation is regarded as change in original Bill of Quantities (BoQ) or additional work items or Specifications changes and when it occurs there is need to modify the contract as required during the performance of contract. It allows to execute unforeseen works without breach of contract or a new contract which will provide proper room from the satisfactory completion and functioning of works and project at end. The contractor is not bound to execute more than contracted work unless contractual provisions bind him. Hence the binding contract documents, financial rules, procurement act and rules etc. should facilitate the variation process

Challenges of Nepalese Construction Industry

Macro Level

- Soaring Construction Demand (both Private and Public Sector)
- Continuing Openness of the Market to International Players (WTO etc.)
- Capacity Building (Financial, Technical and Human Resources, Mechanization and New Technologies)

Micro Level

- Prevailing Procurement Practices
- Project Formulation and Design
- Contracts Administration

Challenges (Micro Level)

Prevailing Procurement Practices

- Qualification Criteria
- Provision for Price Adjustment
- Slicing and Packaging of Contracts
- Excessive low bid
- Provision of Central Data Bank of Contract Details
- Works up to 6 million for user's group
- Contractors Specialization of Works
- Promotion of Domestic Contractors

Project Formulation and Design

- Estimation of Project Period
- Estimation of Project Cost
- Adequacy of Design

Contracts Administration

- Allocation of less budget & Late Payment
- Lengthy Tender Evaluation & Variation Process
- Force Majeure (Band, Fuel and other Construction Material Shortages)

Present Qualification Criteria

- Average Annual Turnover of best 3 years out of

3/5/7/10year = 1.5~2xV/T

- Specific Construction Experience

= at least 1~3 Contracts of estimated value within last 3/5/7/10 years

- Bid Capacity

= Working Capital x (5~10) + Lines of Credit – (40% of) Current Contract Commitment

Present Real Practices

- Make joint venture with other companies to meet the qualification criteria (more than 95% projects)
- Execute the project singly (more than 95% projects)

- Pay1%-7% commission to Joint Venture partners and take the power of attorney from them
- Submit highly qualified but unavailable personnel's bio-data at the time of bidding (including expired, abroad, government officials)

Provision for Price Adjustment

- Sudden Price fluctuation in construction materials and fuel
- Price adjusted only in major construction materials beyond 10% of contracts of construction period less than 15 months
- Project period unreasonably reduced to 15 months just to avoid risk of price escalation
- No Price Adjustment in some projects of contract period more than 18 months/2 years
- No price adjustment beyond 25% (Clause 119(3) of Public

Procurement Regulation 2064)

Slicing and Packaging of Contracts

- No Standard Norms

Excessive Low Bid

Reasons:

- to fulfill the high qualification criteria of forthcoming project / survival in market
- Execution and Acceptance of low-quality works
- Demand / Supply
- Lack of professionalism in the industry
- Provision of Central Data Bank of Contract Details
- Works to user's group (political cadre)
- Specialization of Works
- Promotion of Domestic Contractors

Project Formulation and Design

Contracts Administration

- Allocation of less budget & Late Payment
- Lengthy Tender Evaluation & Variation Process
- Force Majeure (Band, fuel and other construction mateials shortages)

Way Forward

Qualification Criteria – to enable growth

- Set minimum required qualification criteria which is:

P = practical R = realistic R = reasonable

- Average Annual Turnover of best 3 years out of last 10 years = 0.5xV/T
- Specific Construction Experience

at least one Contract of value = 0.5 x estimated value
at least two contracts of value = 0.3 x estimated value
at least three contracts of value = 0.2 x estimated value

- Bid Capacity

= Working Capital x (15~20) + Lines of Credit – (20% of Current Contract Commitment)

Price Adjustment

- Provision of Price Adjustment as per Nepal Rastra Bank Index irrespective of contract period.

Excessive Low Bid

- Rejection of low-quality work and black listing
- Strictly Implementation of bid capacity Provision
- Reasonable Qualification Criteria
- Award Contract who is nearest to the average of Contractors' bid
- Monitoring of proper utilization of Mobilization Advance
- Reasonable Extra Performance Guarantee
- Provision of Milestones

Slicing and Packaging shall be done based on

- past experience
- present bid capacity of contractors
- more scientific and economic

Central Data of Contract Details

There should be a national level central data bank of all contracts details which will help

- to set qualification criteria
- to decide in slicing and packaging
- to evaluate the bid capacity of bidders
- and many more…….

Estimation of Project Period

Project period shall be estimated seriously which is realistic and more practical considering

- Project location
- Accessibility to the Project
- Project start date/Working period

Estimation of Project Cost

- Strict implementation of Clause 9(2) of Public

Procurement Regulation 2064

- District Rates finalized normally in August (off season)
- Normally water is not taken into consideration during cost estimate

Allocation of Less Budget & Late Payment

- Work shall be procured only if there is sufficient budget
- Payment shall be done in time
- Interest rate for late payment shall be more than bank rate

Tender Evaluation and Variation Process

Tender evaluation and variation process shall be done in time

Works to User's Group (Political cadre)

- Equipment hiring shall not allowed
- Contribution of user's group compulsory
- Subletting shall not be allowed
- Proper monitoring necessary

Specialization of Works

- Contractors shall be specialized in particular works in order to be more efficient and professional
- Concern authority shall issue license accordingly

Promotion of Domestic Contractors

- International contractors must have J/V with Nepali Contractors in order to transfer the new technology.
- 10 % price preference to Nepalese Contractors
- Relaxed qualification criteria to Nepalese Contractors

Claims arises when the Contractor believes he has been impeded in some way from works according to the contract.

Causes	Potential Responsibility
Delay in obtaining possession and access to the site	Employer
Delay in obtaining work permits, custom clearance	Employer
Delay in obtaining drawings and instructions	Consultant
Delay of commencement or completion of works by others	Employer
Delay in Payment	Employer
Mismatch in quantities/Variation	Consultant
Design, layout error	Consultant
Extension of Time	Employer/Consultant/ Contractor
Interpretation of specification	Consultant
Unusual Weather Conditions	None
Strike and Civil disturbances	None

Construction Claim Management Phases

1. Claim Prevention
   - The claim prevention process is activated at Pre-tender and Contract Formulation phases of a project. Contract documents project plans and scope of work should include all requirements related with the project because after the award of contract the opportunity to prevent claim comes to an end.

2. Claim Mitigation
   - Construction activities are generally performed in highly sensitive and outdoor environments. It is better to minimize the possibilities of occurring claim all through the progression of the contract. A well-defined scope, responsibilities and risks will help to decrease the possibility of occurrence of claims. Also risk management plans play important roles in the phase of claim mitigation.

3. Claim Identification and Quantification
   - Claim identification can be done by analyzing both the scope of work and the provisions of the contract. Inputs of the claim identification process are the scope of work, contract terms, definition of extra work and definition of extra time requested. Once an activity is identified as a claim, it will be quantified in terms of additional payment or a time extension to the contract completion or other milestone date. In this phase, schedule and critical path analysis should be made in order to calculate the delay of the project. In addition to that, additional direct and indirect costs originated from the claim activity should be calculated.

4. Claim Resolution

- Claim resolution is a step-by-step process to resolve the claim issues. Depending on the resolution terms of the contract, negotiation, mediation, arbitration and litigation processes will be conducted.

Claiming Procedure

- ➢ Start keeping a detailed record simultaneously with occurrence of an event which gives rise to claim
- ➢ Give a notice of intention to claim within stipulated time
- ➢ Act on the Consultant's instructions regarding additional records needed to substantiate the claim
- ➢ Submit a claim with supporting information within the time limit
- ➢ For an event with continuing effect, submit an interim account of claim on regular basis
- ➢ Include an application for payment in addition to actions

Contents of Contractor's claim document:

1. Background
    - ✓ Provide historical data affecting the subject matter of the claim
    - ✓ Make all necessary references to other documents for appreciation of the background
2. Contractual Argument
    - ✓ State the particulars clause or clauses on which the claim is founded
    - ✓ Set out a logical argument in detail so that the Consultant and Employer may understand the claim
    - ✓ Refer to similar known settled claims in the country under similar contracts
3. Supporting Data,Site records, Photographs,Site diaries,Daily Weather Reports
    - ✓ Consultant's site instructions, Working Drawings, Minutes of Meeting, Visitor's Register
    - ✓ Quality control documents, work program, correspondences. Plants, records, Fuel and labor records

4. Financial Comparison

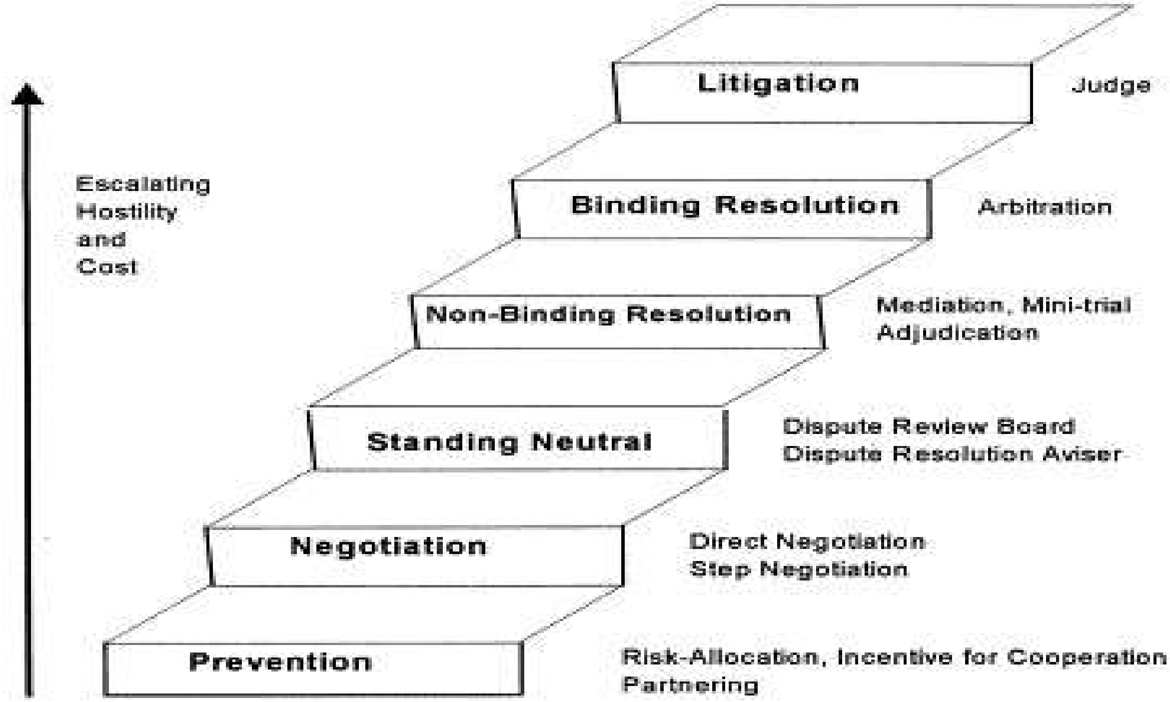
- ✓ Comparison between the cost anticipated by the contractor during the bid with the cost actually incurred
- ✓ Effect of circumstances giving rise to claim

- ✓ Dispute: A 'disagreement' between Employer and the contractor over the payment of money, the adjustment or interpretation of contract terms, any claims arising out of or relating to any aspect of a solicitation, bid, or failure to conduct a solicitation or bid, any decision to award, deny, suspend or cancel, terminate or not renew, any contract or agreement.
- ✓ Dispute Resolution Procedures: Apply to and shall constitute the exclusive procedure for resolution of all claims, disputes, complaints and Dispute Resolution Requests of any kind filed by an Aggrieved Person relating in any way to any agreement entered into by the Vendor.

What is Dispute Resolution

- Dispute resolution refers to the processes by which disputes are brought to an end.
- Dispute Resolution occur through:

1. A negotiated outcome: Parties concerned sort out things themselves
2. A mediated outcome: Parties use the services of an independent mediator to help them arrive at their own agreement, or
3. An arbitrated or adjudicated outcome: An independent arbitrator or court determines how the dispute is to be resolved and makes a binding decision or order to this effect.
4. Dispute resolution or Dispute settlement is the process of *resolving disputes* between parties.
5. The term *dispute resolution* is sometimes used interchangeably with conflict resolution, although conflicts are generally more deep-rooted and lengthier than disputes.
6. Dispute resolution techniques assist the resolution of antagonisms between parties that can include citizens, corporations, and governments.



Forms of Dispute Resolution

- Dispute resolution ranges from informal, non-legally binding methods to more structured legal procedures.
- All of them apply to workplace conflict, and all of them can help organizations avoid lawsuits and other legal battles.
- Some of the most common forms of conflict resolution are negotiation, mediation, arbitration and mediation-arbitration
- All of them focus on solving the conflict with the best interests of all parties involved in mind and avoiding court.
- Simple Dispute/s: Disputes if not too severe, simple negotiation might suffice. With this process, the conflicting parties agree to discuss their concerns with each other openly.
- Suggestions: Parties might share precisely what actions, practices or policies they are upset about and make suggestions about how the dispute/s can be resolved.
- Compromise for Comfort: As part of the dispute resolution, the involved parties typically agree to work together to find a compromise with which they all feel comfortable.
- Informal Process: There will be a meeting between the conflicted parties and a member of the senior management.

Best Practice Dispute Resolution Outcomes

Best Practice should be:

- Quick - the issues should be resolved quickly rather than allowing them to escalate through inaction.
- Fair - all relevant parties should be consulted so that all sides of the story are taken into account.
- Handled sensitively - disputes should, where possible and appropriate, be resolved in a confidential context in order to minimize impact on employees not affected by the dispute.
- Transparent - the procedure should be made known to every employee.
- Dispute resolution procedures should not interfere with the continued operation of the business where possible.
- Continue during Dispute Resolution Process: Any dispute resolution clause in an agreement, contract or policy should require that work is to continue normally during the dispute resolution process subject to any reasonable concerns about health and safety.

Dispute Resolution: Processes

Dispute resolution processes fall into two major types:

- Adjudicative processes: Such as litigation or arbitration, in which a judge, jury or arbitrator determines the outcome.
- Consensual processes: such as collaborative law, mediation, conciliation, or negotiation, in which the parties attempt to reach agreement.

Not all disputes, even those in which skilled intervention occurs, end in resolution.

Dispute resolution is an important requirement in international trade, including negotiation, mediation, arbitration and litigation.

Dispute Resolution: Mediation

- Goal of mediation: It is for a neutral third party to help disputants come to a consensus on their own.
- Rather than imposing a solution, a professional mediator works with the conflicting sides to explore the interests underlying their positions.
- Mediation can be effective at allowing parties to vent their feelings and fully explore their grievances.
- Working with parties together and sometimes separately, mediators can try to help them hammer out a resolution that is sustainable, voluntary, and nonbinding.

Dispute Resolution: Arbitration

- A neutral third party serves as a judge who is responsible for resolving the dispute.

- The arbitrator listens as each side argues its case and presents relevant evidence, then renders a binding decision.
- The disputants can negotiate virtually any aspect of the arbitration process, including whether lawyers will be present at the time and which standards of evidence will be used.

Dispute Resolution: Litigation

- The most familiar type of dispute resolution, civil litigation typically involves a defendant facing off against a plaintiff before either a judge or a judge and jury.
- The judge or the jury is responsible for weighing the evidence and making a ruling. The information conveyed in hearings and trials usually enters, and stays on the public record.
- Lawyers typically dominate litigation, which often ends in a settlement agreement during the pretrial period of discovery and preparation.

Mechanism for Dispute Settlement (PPA-Sec.58)

- Amicable Settlement: Any dispute arising between the Public Entity and the construction entrepreneur in connection with the implementation of the procurement contract shall be settled amicably.
- Arbitration: If the dispute could not be settled through amicable settlement, then the contract agreement should state that the dispute is settled through arbitration as per the prevailing law (Arbitration Act 2055).

Provision Relating to Dispute Resolution

- Dispute Resolution (Rule 129): A procurement contract may provide a mechanism for a resolution of dispute by stating the amicable settlement meetings and decision procedure, application procedure and subject of dispute resolution through amicable settlement as per the section 58 of the PPA 2058.
- Dispute Resolution Through Arbitration (Rule 135): If the dispute could not be resolved through the amicable settlement as per rule 129, shall initiate the proceedings of resolving such a dispute by means of an arbitration in accordance with law in force.

Dispute Settlement & Procedures (SBD)

- Amicable Settlement: The Employer and the Contractor shall attempt to settle amicably by direct negotiation any disagreement or dispute arising between them under or in connection with the Contract.
- Period to refer Arbitration: Any dispute between the Parties as to matters arising in the Contract which cannot be settled amicably within thirty (30) days after receipt by one Party of the other Party‘s request for such amicable settlement may be referred to Arbitration within 30 days after the expiration of amicable settlement period.
- Procedures of Arbitration: Arbitration shall be conducted in accordance with the arbitration procedures published by the Nepal Council of Arbitration (NEPCA) at the place as mentioned in SCC.

Arbitration Act, 2056 (1999)

- "Agreement": A written agreement reached between the concerned parties for a settlement through arbitration of any dispute concerning any specific legal issue that has arisen or may arise in the future under a contract or otherwise.
- "Dispute": A dispute which can be settled through arbitration under Arbitration Act.
- Counter-claim: means a claim made by the Respondent on the Claimants.
- "Rejoinder": A claim to the counter-claim by the Claimants.

"Arbitrator": An arbitrator appointed for the settlement of a dispute and the term also includes a panel of arbitrators

Disputes to be Settled through Arbitration (Sec.3)

- Procurement Agreement has Arbitration Clause: In case any agreement provides for the settlement of disputes through arbitration, the disputes connected with that agreement or with issues coming under that agreement shall be settled through arbitration according to the procedure prescribed in that agreement, if any, and if not, according to this Act.
- File Price in case of concerned parties to a civil suit of a commercial nature which has been filed in a court and which may be settled through arbitration according to prevailing laws, file an application for its settlement through arbitration, such dispute shall also be settled through arbitration.

Number of Arbitrators

- Arbitrator Number/s: The number of arbitrators is as specified in the agreement. In case the agreement does not specify the number of arbitrators, there shall ordinarily be three arbitrators.
- Turn into Odd: In case the number of arbitrators appointed under the agreement is an even one, it shall be turned into an odd one by designating an additional arbitrator chosen by them.

Appointment of Arbitrator

- Appointment of Arbitrator: The process of appointing arbitrators must be started within 30 days from the date when the reason for the settlement of a dispute through arbitration arises.
- In case the agreement mentions the names of arbitrators, they themselves shall be recognized as having been appointed as arbitrators.
- Separate Provision: If agreement has made any separate provision for the appointment of arbitrators, arbitrators shall be appointed accordingly.
- Each Party to Appoint: Each party shall appoint one arbitrator each and the arbitrators shall appoint the third arbitrator who shall work as the chief arbitrator.

Appointment of Arbitrators by Court:

Appointment by Court: The circumstances:

- In case no arbitrator can be appointed upon following the procedure contained in the agreement.
- In case the agreement does not mention anything about the appointment of arbitrators.

CV Details: Must explicitly mention the full name, address, occupation and the field of specialization of at least three persons who can be appointed as arbitrator, and also be accompanied by a copy of the agreement.

Submission of Claims, Counter-Claims, objections or Rejoinders

- Submission of Claims: The claimant shall submit its claim mentioning the details of the subject-matter of the dispute and the remedy sought, along with evidence, and also supply a copy thereof to the other party within the time limit mentioned in the agreement, if any,
- If there is no time limit mentioned then within three months from the date when a dispute requiring arbitration has arisen in case only the name of the arbitration has been mentioned in the agreement without mentioning any time limit, and from the date of appointment of the arbitrator in case the arbitrator has been appointed after the dispute has arisen.
- Objection to Claim: Other party shall submit its objection to it within 30 days from the date of receipt of the claim, unless otherwise provided for in the agreement.
- Counter Claims: In case it submits a counter-claim also, the arbitrator shall provide a time limit of 15 days to claimant submit its rejoinder over such counter-claim. In case a rejoinder is so submitted a copy thereof shall be supplied to the party making the counter claim.
- Time Extension: The circumstances beyond its control, it may submit an application to the arbitrator for an extension of the time limit within 15 days from the date of expiry of the time limit, explicitly mentioning satisfactory reasons for its failure to do so. The arbitrator may, if he/she finds the reasons mentioned in the application to be satisfactory, extend the time limit for not more than seven days.
- Documents in Full: While submitting claims, counter-claims, objections or rejoinders all documents, as well as evidence substantiating them, if any, shall also be submitted.
- Copies to Other Party: Each party submitting documents to the arbitrator in connection with arbitration proceedings shall supply copies thereof to the other party.

Power of the Arbitrator to Determine Jurisdiction (Sec 16)

- Jurisdiction: If arbitrator has no jurisdiction over the dispute which has been referred to him/her for settlement, or that the contract because of which the dispute has emerged is itself illegal or null and void, it may claim so before the arbitrator. The arbitrator shall take a decision on his/her jurisdiction or the validity or effectiveness of the contract before starting the proceeding on the matter referred to him/her.
- Time of Appeal to Court: Any party is not satisfied with the decision taken may file an appeal with the Appellate Court within 30 days from the date of decision, and the decision taken by that court on the matter shall be final.

Arbitrators to Follow Substantive Law

- The Nepal Law shall be the substantive law to be followed by the arbitrator, except when otherwise provided for in the agreement.
- The arbitrator may settle the dispute according to the *principle of justice and conscience* (Ex aqua et bono) or *natural justice* (amiable compositor) only when explicitly authorized by the parties to do so.
- The arbitrator shall settle the dispute according to the conditions stipulated in the concerned contract.
- Arbitrator shall also pay attention to the commercial usages applicable to the concerned transaction.

Arbitrators Decision Time Period

- Arbitrators should take decision as provided for in the agreement.
- If time is not mentioned in the agreement, then the arbitrator shall pronounce the decision ordinarily within 120 days from the date of submission of documents (Sec 24).
- In case any issue requiring arbitration is found to be inextricably linked with any other issue on which the arbitrator cannot pronounce the decision, the arbitration shall not pronounce decision on that issue (Sec 24).
- If arbitrator cannot take a decision, the arbitration must inform the concerned parties accordingly.
- The concerned party may file a complaint to the Court within 35 days from the date of receipt of a notice as per the prevailing law.

Matters to be mentioned in Decision (Sec. 27)

- Arbitration Issues: Brief particulars of the matter referred to for arbitration.
- Jurisdiction of Arbitration: Grounds for deciding that the matter falls under the jurisdiction of arbitration.
- Reasons & Grounds of Decision: Arbitrator’s decision, and reasons and grounds for reaching that decision.
- Determine the Amounts/ validate Issues: Claims which must be realized or amounts which must be compensated.
- Interest / Additional Interest: Interest on amount to be realized, and the additional rate of interest to be charged with after the expiry of the time limit for implementing the decision of the arbitrator in the event of the limit mentioned in section.
- Place and date of decision.

Decision Invalidation Circumstances (Sec. 30)

- Time of Appeal to Invalidate: Any party dissatisfied with the decision taken by the arbitrator may, if one wishes to invalidate the decision file a petition to the High (Appellate) Court along with the related documents and a copy of the decision within 35 days from the date the decision heard or notice received thereof. Petition shall also supply a copy of that petition to the arbitrator and the other party.
- Petitioners to Prove: In case a petition is filed in the High (Appellate) Court the petitioner need to prove that the arbitration decision contains matters that invalidate decision or issue an order and need to have a fresh decision be taken

Implementation of Award (Sec.31)

- Implementation Time of Decision: Concerned parties shall be under obligation to implement the award of the arbitrator within 45 days from the date when they receive a copy thereof.
- Implementation of Award by Court: In case a award cannot be implemented within the time limit as above, the concerned party may file a petition to the District Court within 30 days from the date of expiry of the time limit prescribed for that purpose to implement the award.

- In case such a petition is filed, the District Court shall implement the award ordinarily within 30 days as if it was its own judgment.

Cost of Arbitration Proceedings/ Arbitrator's Remuneration

- Fixed Amount: Parties seeking arbitration must pay to the arbitrator the amount fixed their in consultation with parties for conducting the arbitration proceedings.
- Proportionate Expenses: Each party shall bear the expenses required for the arbitration proceedings in the proportion prescribed by the arbitrator taking into account the relevant circumstances.
- Arbitrator's Remuneration: Shall be as prescribed in the agreement.
- If remuneration is not mentioned in the agreement: Concerned parties shall pay the remuneration fixed by the arbitrator in consultation with them. Paid as a full payment or advance payment.

Dispute Settlement Perspectives in Nepal

- In Nepal, the provision of arbitration was introduced in 1957, while the Development Committee Act 1956 was amended.
- The provisions were confined up to the dispute to which the Development Broad is a party to the contract.
- Real practice of dispute settlement through arbitration was stated after the enhancement of Nepal Arbitration Act 1981 (Now, Arbitration Act 1998)- This is based on UNCITRAL model law.
- Nepal has very short history of modern arbitration concept. Before enactment of the Arbitration Act 1981, arbitration was carried out through a local celebrity when it becomes necessary to settle the disputes between the villagers and it has been in practice for a long time. Slowly business people felt necessary of a quick and cheap methodology of settling disputes and came up with arbitration proceedings.
- The new act stated that the disputes arising out of the agreement made with foreign donor agency shall be settled through arbitration, So, some examples of excessive delay up to four or five years for the settlement of disputes.
- It was felt as expensive as well as cumbersome and has adverse effect on the main purpose of arbitration.
- To solve the problems associated with arbitration and to assist in the development of adequate infrastructure, Nepal Council of Arbitration (NEPCA) was established in 1991 through initiation of non - government sector.
- It is important to understand the problem related to dispute and try to mitigation it in construction contract in the context of Nepal.

Causes for Emergence of Disputes in Nepal

- A dispute arises when a demand is made by a party and denied by the other and the contradiction is not accepted by the demanding party (either employer or contractor).
- Dispute originates due to disagreement on a decision or action taken by one party on the ground of effect to be borne by the other as consequence of the decision.
- In construction projects in all sector of development either completed or ongoing have suffered from time and cost overrun.
- A major reason for this is poor management of contract resulting in disputes leading to intermittent stoppage of works or slow progress or even abandonment of work requiring fresh call of tenders to engage a new contractor for execution.

Major Causes of Disputes in Nepal (Road Project/s)

Generally: four areas of disputes- contract document, force measure, timely action, and project characteristics

- Change of material Source
- Inadequate design and site information
- Commencement and Delay information
- Unforeseen physical / Site condition resulting to variation
- Strikes, Bandh, Riot (disturbance) or Disorder
- Delay in decision making and settlement of dispute
- Possession of site and Access to site
- Unusual weather condition and Inflation
- Unavailability of fuel and Construction material

Problematic Areas for the claim and disputes

- Engineer doesn't work impartial and do not fulfill their responsibility promptly.
- Contractors generally do not fulfill or are reluctant to perform contractual obligations.
- Employers are not prompt in decision making for any problems.
- Employers are not very serious toward fulfillment of their contractual obligation.
- Incorrect and inconsistent Drawing.

Dispute/ Claims Resolution Practices in Nepal

- Due to small size of construction industry of Nepal, the problems associated with disputes are not so similar to developed countries.
- A common problem generally found in Nepalese context is launching many claims but abandoned at last.
- Main reason of such abandonment is due to Employer dominated contact documents, Contractor's right minimally protected and low level of knowledge regarding contractual rights and obligation among the contracting parties.
- Neither employer nor contractor gives adequate attention about status of contract documents before entering into the contract Which, generates adequate ground to gives rise of many problems during contract execution.

- Due to no adequate provision incorporated in contract to tackle the probable situation liable to dispute, its resolution seems very difficult.
- Party-initiating disputes, a contractor has only two options these are:
  - Abandon the claims or disputes or
  - Go to litigation in court.
- Court Process being contractor is compelled to choose former one other very time consuming wise he suffers of payment delay of due amount.
- The main reason of delay was improper provision of disputes resolution in contract clause.
- The clause prevented to enter into arbitration process unless the work is completed.

Recommendation to Reduce Disputes in Nepal

- Proper Site Investigation: Claims arising from unforeseen physical conditions can be reduced by comprehensive site investigation during the phase of details design preparation.
- Risk and Budget: The employer / executing agency should prepare to bear all risk and allocate with add proper budget in contingencies for the issues arises from disputes / claims.
- Realistic BoQ& Periods: Consultants (i.e., Engineer) should ensure that BOQ and contract periods, which specified in the bid documents, are realistic and take account of existing site conditions.
- Facilitate regular execution work and make prompt decisions: Consultants (i.e., Engineer) should provide any information with corrected drawings timely on site to facilitate regular execution work and make prompt decisions regarding the technical and contractual issues by taking employer approval where necessary in specified time as required.
- Organize a kick off meeting: Consultants (i.e., Engineer) should organize a kick off meeting with contractors to confirm material availability, constructability and other constraints flagged up by contractors before their site mobilization.
- Better to establish a dispute settlement unit should be established to amicably (negotiation) settle any disputes before referring to contractual authority.
- Regular Management and Site Meetings involving employer, engineer and contractor. The time periods may be once a week, two weekly or monthly meetings among them.
- Recording of every minute of meeting and issues for the future reference regarding EoT, Price Adjustment, determine compensating events and even force majeure etc.
- Strictly adhere the Work Schedule and make contract administrator / coordinator responsible for non-performing of contract and punishing the contractor as per agreement.

ISF Related

Causes of Poor Collection of ISF	Plan of Actions
Poor service delivery and hence little incentive to pay ISF	Enhancement of irrigation service delivery through various models of irrigation management transfer. Transparent and well-publicized regime of setting tariffs and charges which are linked to the level of irrigation service Agreed mechanisms, involving WUAs, DoWRI and local government, for monitoring irrigation service performance
	Local implementation (of nationally agreed formulas) to ensure local conditions reflected in irrigation fees Ensure equitable water distribution to encourage all WUA members to pay
Lack of legal authorization to WUA for enforcing rules of irrigation management and ISF collection	Register WUAs with local government for governance support Establish bye-laws and other supporting legislation/rules to preserve principles of payment for irrigation service, and support WUAs in institutionalizing irrigation rules and concept of “service–for– fee”

Inadequate capacity of WUAs (or WUGs)	Enhance capacity of WUAs (or WUGs) for all aspects of ISF collection
Payment avoidance by some influential members	Define the legal basis of WUAs for setting fees in relation to service and its collection A graduated system of sanctions should be in place to oblige payments Legal action to collect arrears
Lack of incentive to pay ISF	Increase the incentive to pay by ensuring adequate levels of service provision At the farm level, have clear agreement about the set of exceptional circumstances under which ISF may be wave dor reduced
Current system of fee collection involves high cost and low collection efficiency	Involve WUAs, with local knowledge and presence, in fee collection on an incentive-earning basis Create legal and procedural basis for delegated fee collection by WUAs/local government agencies Allow partial retention of fee-by-fee collectors to provide incentives for improved collection

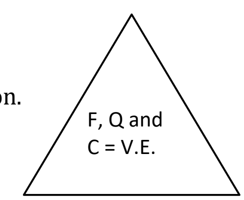
What is the value management and how can you effectively use it in Nepalese context for better utilization of resources?
What are the elements of value engineering? Are these examples of Value engineering application in Nepal? Discuss its possible application in Nepal.

Definition

"Systematic application of recognized techniques, knowledge and skills to identify the function of a project, service or a process to improve performance, quality and or life cycle cost there by establishing a "true worth" for that function.

Components

- An organized review to improve value by using cross functional team.
- A functional oriented approach to identify and analyze the essential function.
- Creative thinking using recognized techniques to explore alternative.
- Judgement thinking finally to arrive at final decision.



Thus,

- Value engineering is a discipline comprising a series of techniques aimed at an organized, systematic efforts directed at analyzing functions of items, products, equipment, process and procedures for the purpose of accomplishing all the required functions at the lowest total cost.
- Unnecessary cost built in design will have to be cut.Value can be increased either by increasing the function or reducing the cost.
- Functional balance between cost, performance and reliability by VE review.
- The goal of VE is to ensure a design that meets the owner's required function at the most reasonable life cycle cost.
- All designs have unnecessary costs - project is usually formed and designed under pressure of meeting with the deadlines the designer will not be able to review it for unnecessary cost.
- The designer must understand that unnecessary cost in a design are not a reflection on his abilities as a professionals, but rather a management problems that needs to be addressed.
- Poor and wrong decisions can be made under the competing pressures of time, budget and quality. As a result all projects are likely to include unnecessary costs.

The reason for poor value occurs:
The challenge is to cut unnecessary costs or to keep at minimum level. Endless reasons for poor value in designs.

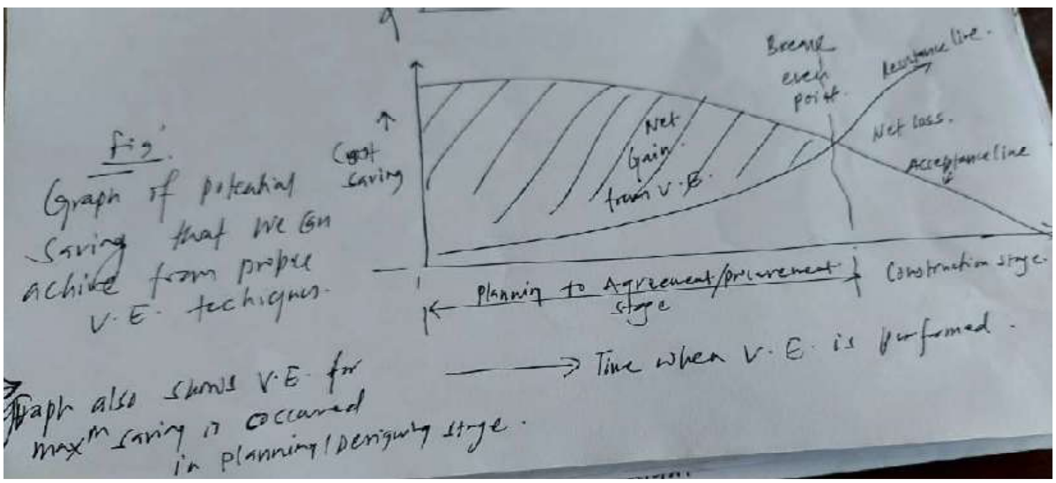
Approach/Method of value increasing

- Improve utility with no change in cost.

- ➢ Retain the same utility from less cost.
- ➢ Combine improve utility and less cost.

Importance/Benefit of V.E. and effect

VE is best utilised as a team approach to provide the optimum value on a project. This means that the whole project team should be involved, from the client right through to the supply chain. In the value engineering, the project is split into its elemental function and alternative methods of accomplishing each function are developed. Each method is carefully evaluated and refined until a workable low-cost method of accomplishing each function is found. Eventually a final course of action is selected that would a achieve the best possible solution for the entire project.



- ➢ Investing time and money in early stage, planning and designing of project is a key source in project delivery. Bringing together, cross functional teams from the government and contractor early strategic planning can avoid the alteration that leads to 60% of project delays.
- ➢ An efficient delivery can create a saving of much as 25% of the new project or 15% saving on total infrastructure and very fruitful for Nepal because various infrastructure project including roads and bridges have been suffering from implementation delay. GoN has to adopt sophisticated procurement, streamline permit approvals and land acquisition, lean construction model to reduce the clogs and bottlenecks and achieve the unprecedented saving.
- ➢ Typically, V.E. study may generate recommendation to cut 10 to 30% project construction costs. The designer usually accepts about half of these recommendation provide saving at least 5% project cost. The cost of value engineering effort including any redesign is usually less than 10% of total saving.
- ➢ VE can also be associated with maximising value, not just reducing costs (cutting upfront project costs is not VE!). VE examines key solutions to extract any unwanted waste, such as water, energy, time, maintenance etc. and reduce life cycle costs whilst providing better function, quality and sustainability.

Value engineering application in Nepal

The use of value engineering in the public sector of construction has been fostered by legislation and regulation, but the approach has not been widely adopted in the private sector of construction. In public sector the fee for desired design services is tightly monitored against the "market price" or may even be based on the lowest bid for service. Such a practice in the setting professional's fees encourages the design professionals to adopt known and tried designs and construction technologies without giving much tough to alternatives that are innovative but risky.

- Design and Built contract for construction of bridges in department of roads in various location in the country.
- Design and Built contract for construction of Airport like Gautam buddha airport.
- Design and supervision consultant by various agencies like DoR, DWRI, CAN etc. in which owner check the value engineering by various alternative.
- EPC contract for construction of Tunnel like Sidhababa tunnel and tunnel in fast track.
- Various standard design guideline/manual which various department such as PDSP manual for irrigation.
- Standard design/drawing of road elements/traffic sign manual/super structure bridge design drawing/multicell culvert etc.
- Provision for field visit before submission of bid in bidding document or RFP to contractor or consultant.
- Provision for pre bidding meeting in contract administration etc.
- Provision for variations and claims.
- Provision for dispute resolution in contract clauses.

These all are directly or indirectly help us in various steps from project planning to hand over but due to various roadblocks/reason our infrastructure field cannot able to use optimum level of value engineering. These are

- Lack of information, usually caused by shortage of time. Too many decisions are based on feelings rather that facts.
- Wrong beliefs, insensitivity to public needs or unfortunate experiences with products or process used is unrelated prior application.
- Habitual thinking, rigid application of standards, customs and tradition without consideration of changing function technology and value.
- Risk of personnel loss. The ese and safety experiences in adherence to established procedure and policy.
- Reluctance to see advice, failure to admit ignorance pf certain specialized aspects of project development.
- Unwilling to lose high opportunity through variation and claims.
- Negative attitude, failure to recognize creativity or innovation.
- Over specifying in document.
- Outdated standard/norms and technology
- Lack of professional expert team in the project.
- Poor human relation, lack of good communication, misunderstanding etc.

Value engineering is a process that establishes specific engineering objectives to maximize the efficiency of design and cost reduction. Generally, value engineering focuses on the best design from among available options. It is tactical tool.

त्यसैले पूर्वाधार क्षेत्रमा मुलुकको कुल बजेटको करिव एक तिहाई बजेट बिनियोजन (विदेशी सहयोग/ऋण को अंश धेरै हुने) हुने र यसको प्रभावकारी तथा किफायती परिचालन बाट नै देशको दीर्घकालीन लक्ष्य,SDG का लक्ष्यहरु हासिल गर्नुपर्ने भएकोले स्रोतसाधनको कमि भएको हाम्रो जस्तो मुलुकमा valueEngineering को महत्वलाई सम्पूर्ण सरोकारवालाले बुझ्नुपर्ने देखिन्छ ।

Technique used in Value engineering for cost reduction.

- Selection of project
- Investigate the project/information gathering
- Creative thinking in alternative.
- Cost analysis
- Pareto rule/analysis i.e., specifies 80% of consequences come from 20% of the causes.
- Basic and secondary functions.
- Cost and worth
- Functional analysis
- Creating thinking through brain storming in various alternative
- Life cycle costing
- Criteria weighting
- Analysis and ranking of alternative values free
- Weighted value tree etc.
- Identify the most valuable one
- Agree a plan for continuity for development, construction etc.

Value engineering by contractor

- Study the project before bidding and determine the effect on topography, geology, climatic, labor supply etc.
- Careful review of bidding document/design drawing/BoQ/specification and contract clause etc.
- Active participation in pre bidding meeting.
- Use of substitute cost reduction.
- Payment of bonus to key personnel for better production
- Use of radio as the means of communication.
- Adoption of realistic, safety, healthy practice in project.
- Practice to holding periodic conference/meeting/workshop etc.
- Desirability of improving shop and servicing facilities for maintenance of costly equipment.

- ➢ Desirability of appropriate sub-contracting.
- ➢ Use of appropriate technology and local resources in efficient manner.
- ➢ Mobilization of appropriate project management team/cross functional team.

Value management

- ➢ Value management is a process that uses consensus driven, collaborative decision making to achieve optimal design while controlling the development of a project in accordance with all stakeholders' needs.
- ➢ Provides a structured framework in which requirements are evaluated against the means of achieving them as the project develops thereby insuring that money and effort is spent where it is most needed and best value for money is achieved.
- ➢ VM is primarily about enhancing value and not cutting cost, cost is reduced as a by-product of VM.
- ➢ VM embraces the whole value process and includes value planning, value engineering and value reviewing and includes; determining the functional requirements of the project or its parts, identifying alternatives and examining cost and value of each alternative to enable the best value selection.
- ➢ Value management should occur before definite design is in place.
- ➢ It is essentially a strategic tool.
- ➢ It is multidisciplinary structural framework which focused on value rather than cost, seeking to achieve an optional balance between time, cost and quality.
- ➢ It embraces value engineering, value analysis and value reviewing.

VM and cost reduction

- ➢ VM is positive, focused on value rather than cost, seeking to achieve an optimal balance between time, cost and quality.
- ➢ VM is structured, auditable and accountable
- ➢ VM is multi-disciplinary, seeking to maximize the creative potential working together.

Life cycle costing (capital costs, operating costs includes staffing, energy consumption, maintenance, cleaning, insurance etc and disposal cost) is a vital element when seeking to optimize value for money.

VM aims to eradicate the need for late changes, VM should not encourage them.

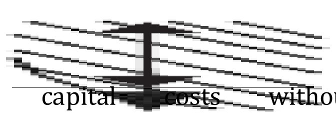
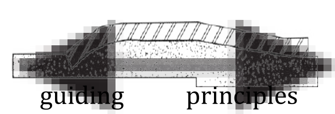
The first review of VM should include - list of objectives identified, objective hierarchy by ranking, feasibility of options and valuable option and selection of most promising option.
Second review should include - review of validity of the objectives, evaluate feasibility of options, examine most promising option, develop a project brief based on most promising option, program for developing the project.

Third review could be during design development (30-40% of design complete) and that include - review project requirements and objectives, check key design decisions taken are relevant, review key decisions against project brief, evaluate options, identify and develop the most valuable one to enhance value, agree a plan for the continued development of the design.

Who to involve:

- The value manager - client's professional advisor, project manager, construction manager
- The project team
- An external team

Differences between Value Engineering and Value Management

Characteristics	Distinguishing between Value Engineering and Value Management *	
	Value Engineering	Value Management
Typical objectives	 Reducing capital costs without compromising quality or performance. Selecting the best option (satisfying all requirements at lowest cost) from a range of options. Choosing between component types such as structural steel or reinforced concrete.	 Developing guiding principles (including principles to achieve best value for money) for planning and design at the briefing stage of projects. Selecting the best concept design options from a range of options. Developing proposals to enhance value

		for money at concept or detailed design stage. Resolving planning and design issues.
Typical focus	"Hard" – technical focus – physical building or component parts.	"Soft" – concepts, "people-activities", preferences.
Stage of project development	There is likely to be at least a concept design, and more likely, some detailed design work. In some cases, design work may be complete or nearly complete. Many Value Engineering studies are undertaken during construction stage, especially when projects are running over time and over budget.	Most likely to be at the early stages of project development, even before a project brief has been prepared.
Participants	Strong technical focus.	Broad participation by stakeholders from management, strategic planning to operational.
Number of participants	Normally tighter in numbers, 8 to 15.	Typically 15 to 25 but sometimes up to 40 or 50 people.
Function analysis	Conventional function analysis of individual components.	Primary purposes; beneficial outcomes, important characteristics that must be achieved – at the "whole entity" level. More detailed function analysis where required.
Cost comparisons	Can generally be more precise in terms of capital and recurrent costing and models.	Possibly indicative, generally comparative (greater than/less than) if costs are even conceivable – eg 50% more than this one. There might not even be a budget at the early stages. The Value Management workshop can provide the basis for establishing one.

This table presents examples that are sometimes used to distinguish Value Engineering from Value Management in the Singapore construction industry. It is important, in reading the table, to recognize that there is no universally agreed distinction between Value Engineering and Value Management, neither in the international literature, nor in international practice and many practitioners and authors use the terms Value Engineering ~~and Value Management~~ interchangeably and synonymously. However, if there is to be difference in use, then the above examples might be helpful in making the distinction.

Value Analysis (VA)

It describes a value study of a project that is already built or designed and analyzes the project to see it can be improved.The purpose of VA is to give a second look to the design of product with the aim to reduce cost without reducing its value. VA is applied after all aspects of design is ready or project completed by a separate team not involved in the design or implementation of a project.

Value analysisinvolves the implementation of a set of techniques relating to cost reduction and cost prevention to the existing product to improve its value. On the other hand, **Value Engineering (VE)** is the implementation of a similar set of techniques relating to a new product at the time of its design. Difference of VA and VE are given below.

BASIS FOR COMPARISON	VALUE ANALYSIS	VALUE ENGINEERING
Meaning	Value Analysis is a cost reduction technique applied to the existing product with the aim of enhancing its worth.	Value Engineering is a technique used before the product gets approval for fabrication.
Nature of Process	Remedial Process	Preventive Process
Applied when	After the product is introduced.	At the design stage
Objective	To get better optimized commercial output.	To get better engineering results.
Worked Out	With the help of knowledge and experience.	With the help of specific technical knowledge.
Ensures	Elimination of unnecessary cost	Prevention of unnecessary cost
Change	May change the existing stage of the product or operation	Changes made by value engineering are implemented at initial stages only.

Approaches (positive and negative) of Value engineering/management to Client/Designer (Consultant) and Contractor.

Party	Positive benefit/Pros	Negative /Pons
Client/Owner/Promotor	➢ Improvement in value, risk reduction ➢ Reduction in capital, o and m cost ➢ Improvement in delivery dates.	➢ Already included in service contract. ➢ Uncertain about process ➢ Thinking of increased risk through innovation.
Designer/Consultant	➢ Reduction of risk exposures ➢ Innovation and better design ➢ Integration of design with construction and safety ➢ Financial gains through additional fees ➢ Marketing benefits for new methods/techniques	➢ May owner criticizing in design capacity ➢ May reduce high quality of element ➢ May be expensive ➢ Risk may increase through innovation
Contractor	➢ Incentive/bonus through sharing of savings ➢ Improved relations ➢ Improved buildability and safety ➢ Expansion of experience	➢ Increase risk with new design ➢ Process may delay ➢ Unwilling to loose high opportunity through variations and claims.

	➢ Reduction of contract period/overhead ➢ Marketing benefits for new methods and techniques ➢ Increasing company "Goodwill"	

8. Environmental Impact Assessment

नेपालमा वातावरणीय अध्ययन र प्रचलित नीति र कानुनहरु

आधार

- ➢ United Nations Conference on Human Environment, 1972 (5-16, June) called Stockholm's Convention
- ➢ Convention on Environmental Impact Assessment in a Transboundary Context, 1991 (Espoo. Finland)
- ➢ United Nations Conference on Economic Development (UNCED)
- ➢ वातावरण प्रभाव मूल्यांकन सन् १९५० को दशक देखि नै वातावरण चेतना वृद्धिको एक भागको रुपमा विश्वब्यापीरुपमा प्रचारप्रसार भएको
- ➢ अमेरिका मा सन् १९६९ देखि EIA ले औपचारिक प्राथमिकता पाए संगै यो प्रक्रिया संसारमा प्रचलनमा आएको
- ➢ राष्ट्रिय संरक्षण नीति,२०४५
- ➢ सुर्य प्रसाद शर्मा ढुंगेल द्वारा सर्वोच्च अदालतमा गोदावरी मार्बल सहित सरकार बिरुद्ध दायर भएको रिट)WP ३५/१९९२ (मा भएको फैसला
- ➢ नेपाल वातावरणीय नीति तथा कार्ययोजना,२०५०
- ➢ राष्ट्रिय वातावरणीय प्रभाव मूल्यांकन मार्गदर्शन,२०५०
- ➢ वातावरण संरक्षण ऐन,२०५३ र नियमावली,२०५४
- ➢ राष्ट्रिय वातावरण नीति,२०७६
- ➢ वातावरण संरक्षण ऐन,२०७६ र नियमावली,२०७७
- ➢ हाल सम्म करिव ३५० देखि ४०० वटा आयोजनाको EIA र १५०० देखि २००० वटा IEE स्वीकृत भएको

राष्ट्रिय वातावरण नीति ,२०७६

- ➢ नेपालको संविधानको धारा ३० ले प्रत्येक नागरिकलाई स्वच्छ वातावारंमा बाँच्न पाउने मौलिक हकको व्यवस्था गरी यसको संरक्षण र व्यवस्थापनलाई राज्यको उच्च प्राथमिकतामा रहेको
- ➢ वातावरण संरक्षण बहुआयमिक राष्ट्रिय तथा अन्तरदेशीय बिषय रहेको हुँदा नेपालले वातावरण सम्बन्धि विभिन्न अन्तर्राष्ट्रिय सन्धि सम्झौतामा गरेको प्रतिवद्धता
- ➢ दिगो विकासको अवधारणा अनुरुप वर्तमान र अंतर्पुस्ता समन्यायका लागि वातावरणीय स्रोतमाथि न्यायोचित पहुँच र तिनको बुद्धिमतापूर्ण उपयोगको प्रत्याभूति गर्नु राज्यको दायित्व
- ➢ विकासका सबै आयामहरुमा वातावरणीय चासोलाई मूलप्रवाहीकरण गर्ने
- ➢ विधुतीय सवारी साधन ,हाइब्रिड सवारीसाधन वा हाईड्रोजन इन्धनबाट चल्ने सवारीसाधन जस्ता नवीनतम तथा स्वच्छ उर्जा खपत गर्ने सवारी साधनहरुको प्रयोगलाई प्रोत्साहन गरिने
- ➢ विकास आयोजना हरुको सबै चरणमा वातावरणीय पक्षलाईआन्तारिकिकरण गरिने
- ➢ विकास आयोजना बाट वातावरण र समाजमा पर्ने प्रतिकुल प्रभावलाई न्यूनीकरण र अनुकूल प्रभावलाई विस्तार गरिने
- ➢ विकास आयोजनाबाट सृजित प्रतिकूल वातावरणीय प्रभावमा परेको समुदायलाई न्यायोचित क्षतिपूर्तिको व्यवस्था गरिने

- वन क्षेत्रमा सार्वजनिक विकास निर्माणको कार्य गर्दा वन क्षेत्र घटे वापत प्रभावित समुहलाई क्षतिपूर्तिको ब्यबस्था मिलाउने
- भौतिक पूर्वाधारको निर्माण गर्दा वातावरणमैत्री संरचना निर्माण गरिने
- राष्ट्रिय प्राथमिकता प्राप्त योजनाको लागि राष्ट्रिय वन प्रयोग गर्ने सम्बन्धि मापदण्ड सहितको कार्यविधि बमोजिम विकास निर्माण कार्य गरिने
- Payment for Ecosystem and Polluter pay Principle लाई आत्मसात गरिने

Environmental Impact Assessment (EIA)

- EIAcanbedefinedasthestudytopredicttheeffectofaproposed activity/projecton theenvironment.
- EIA is a decision-making tool, which compares various alternatives for a projectandseeksto identifythe onewhichrepresentsthebest combinationof economicand environmentalcosts and benefits.
- EIA systematically examinesbothbeneficialandadverseconsequencesof the project and ensures that these effects are taken into account during project design.
- It helps to identify possible environmental effects of the proposed project, proposes measures to mitigate adverse effects and predicts whether there will be significant adverse environmentaleffects, even after the mitigationis implemented.
- 'It is an amalgamation of Artand Science ofidentifying/predicting and evaluating the results ofinteractions betweenenvironmental variables and human activities in nature."
- EIAas anArt or aManagementTool Reflectssensitivitytowardsnature
    - Carriesoutenvironmentalanalysis of actions
    - Ensurescompliancewiththepolicyandlegalprovisions
    - Influencesdecision-makingprocess
- EIAas a Science deals withmethodologies and techniques for identifying, predicting and evaluating the environmental impacts associated with a particular proposal.
- Systematic identification and evaluation of the impacts on the environment caused by a proposed project.
- Formalprocesstopredictenvironmentalconsequences/impactsof a proposed plan/project.
- Donepriortothedecisiontomoveforward.
- Environmental consequences/impactsincludechangesinphysical, ecological & socio-economiccomponentsof environment.
- Donebefore,duringandaftercompletingproject.
- Ensures public consultationand participationof affected community.
- EIAmustbemadeforalldevelopmentprojects.
- Predict environmental impacts at anearly stagein projectplanning anddesign, findwaysandmeanstoreduceadverseimpacts,shapeprojectstosuitthelocal environment andpresent thepredictions andoptions todecision-makers.
- Designed to be a constructivetool which ensures that the project does not give rise to problemsaffectingany aspect ofthe environment.
- Helpstoensurethatdevelopmentimprovesthewayoflifeforthepeople affected,withoutdamagingthenaturalsurroundings.
- Helpstojudgeenvironmental performanceoftheproponent.
- Providesinputsintodecision-making.
- By considering the environmental effects of the project and their mitigation early in the project planning cycle, environmental assessment has many benefits,suchasprotectionofenvironment,optimumutilizationof resourcesand saving oftime and cost ofthe project.
- Benefits of integrating EIAhave beenobserved in all stages of aproject, from exploration and planning, through construction, operations, decommissioning and beyond site closure.
- Properly conducted EIA also reduces conflicts by promoting community participation, informing decision makers and helping lay the base for environmentally soundprojects.
- Sometimesadegreeofdamageisinevitable.
- In this case an EIA should find ways of reducing or compensating for the damage (National Road Authority, 2008). This is by the use of mitigation measures.
- An application of Environmental Assessment (EA) Study is legallyrequiredpriorto the implementation ofthe project(HMG,1997).

ObjectivesofEIA

- **To ensurethattheenvironmentalconsiderations areclearlyaddressedand incorporated into the developmentand decision-making process.**
- **Toanticipateandavoidorminimizetheadversebiophysical,socialand otherrelevanteffects ofdevelopmentproposals.**
- **Toprotect theproductivity and capacity of natural systems and theecologicalprocesses.**
- **To promote development that is sustainable and optimize resources use as well as management and opportunities.**

MajorFunction ofEIA

- **Identifypotentialenvironmentalimpacts**
- **Examine thesignificanceoftheenvironmentalimpacts**
- **Assist whether or not the impact can be mitigated**
- **Recommendpreventiveandcorrectivemeasures.**
- **Assistdecisionmakerstodeterminewhethertheparticulardevelopmentproject shouldgoahead**
- **Provideinformation todecisionmakersandotherintegrated partiesabout environmentalimplications**

वातावरण संरक्षण ऐन ,२०७६ र नियमावली २०७७

प्रस्तावना :स्वच्छ र स्वस्थ वातावरणमा बाँच्न पाउने प्रत्येक नागरिकको मौलिक अधिकारको संरक्षण गर्न, वातावरणीय प्रदुषण वा ह्रास वाट हुने क्षति वापत पिडितलाई प्रदुषक बाट क्षतिपुर्ति उपलब्ध गराउन ,वातावरण र विकासबीच समुचित सन्तुलन कायम गर्ने, प्रकृति ,वातावरण र जैविक विविधतामा पर्ने प्रतिकुल वातावरणीय प्रभाव न्यूनीकरण गर्न तथा जलवायू परिवर्तनको चुनौतीलाई सामना गर्नको लागि वातावरण संरक्षण सम्बन्धि प्रचलित कानूनलाइ संशोधन र एकीकरण गर्न जरुरी भएको ।

वातावरण संरक्षण ऐन,२०७६

परिभाषा:

- **वातावरण:** प्राकृतिक, साँस्कृतिक र सामाजिक प्रणाली, आर्थिक तथा मानवीय क्रियाकलाप, यिनका अवयवहरु तथा यी अवयवहरुको बीचको अन्तरक्रिया तथा अन्तरसम्बन्ध सम्झनु पर्दछ ।
- **प्रस्ताव:** विद्यमान वातावरणीय अवस्थामा परिवर्तन ल्याउन सक्ने किसिमको विकास कार्य, भौतिक क्रियाकलाप वा भू-उपयोगको परिवर्तन गर्ने कुनै योजना, आयोजना वा कार्यक्रम संचालन गर्ने सम्बन्धमा तयार गरिएको प्रस्ताव सम्झनु पर्दछ ।
- **प्रस्तावक:** स्वीकृतिको लागि निवेदन दिने वा प्रस्ताव कार्यान्वयन गर्न स्वीकृत प्राप्त व्यक्ति वा सरकारी, अर्धसरकारी, गैरसरकारी, निकाय वा संस्था सम्झनु पर्दछ ।

संक्षिप्त वातावरणीय अध्ययन) BriefEnvironmentalStudy ,BES : (कुनै प्रस्तावको कार्यान्वयन गर्दा सो प्रस्तावले वातावारणमा उल्लेखनीय प्रतिकुल प्रभाव पार्ने वा नपार्ने सम्बन्धमा यकिन गनुको साथै त्यस्तो प्रभावलाइ कुनै उपाय द्वारा निराकरण वा न्यूनिकरण गर्नका लागि अवलम्बन गरिने उपायको सम्बन्धमा संक्षिप्त रुपमा गरिने अध्ययन वा मूल्यांकन सम्झनुपर्छ ।

प्रारम्भिक वातावरणीय परिक्षण) IEE : (कुनै प्रस्तावको कार्यान्वयन गर्दा सो प्रस्तावले वातावारणमा उल्लेखनीय प्रतिकुल प्रभाव पार्ने वा नपार्ने सम्बन्धमा यकिन गनुको साथै त्यस्तो प्रभावलाइ कुनै उपाय द्वारा निराकरण वा न्यूनिकरण गर्नका लागि अवलम्बन गरिने उपायको सम्बन्धमा विश्लेषणात्मक रुपमा गरिने अध्ययन वा मूल्यांकन सम्झनुपर्छ ।

वातावरणीय प्रभाव मूल्यांकन) EIA : (कुनै प्रस्तावको कार्यान्वयन गर्दा सो प्रस्तावले वातावारणमा उल्लेखनीय प्रतिकुल प्रभाव पार्ने वा नपार्ने सम्बन्धमा यकिन गनुको साथै त्यस्तो प्रभावलाइ कुनै उपाय द्वारा निराकरण वा न्यूनिकरण गर्नका लागि अवलम्बन गरिने उपायको सम्बन्धमा विस्तृत रुपमा गरिने अध्ययन वा मूल्यांकन सम्झनुपर्छ ।

पुरक वातावरणीय प्रभाव मूल्यांकन :एक पटक स्वीकृत भैसकेको वातावरणीय प्रभाव मूल्यांकन सम्बन्धि प्रस्तावमा आंशिक रुपमा भौतिक पूर्वाधार, डिजाइन वा स्वरूप परिवर्तन गर्न, संरचना स्थानान्तरण वा फेरबदल गर्न, वन क्षेत्र थप गर्न वा आयोजनाको क्षमता वृद्धि गर्नको लागि पेश भएको प्रस्ताव उपर पुनःगरिने वातावरणीय प्रभाव मूल्यांकन सम्झनु पर्दछ ।

रणनीतिक वातावरणीय विश्लेषण) SEA : (परिभाषा नभएको

वातावरण संरक्षण ऐन,२०७६

- **वातावरणीय अध्ययन (दफा ३-२):** संघ, प्रदेश र स्थानीय तह
- **सार्वजनिक सुनुवाई (दफा ३-५):**वातावरणीय अध्ययन प्रतिवेदन तयार गर्दा ***(BES, IEE & EIA)***
- **मापदण्ड एवं गुणस्तर कायम गर्नुपर्ने (दफा ६):**

  १. प्रस्तावकले यस ऐन बमोजिम वातावरणीय अध्ययन प्रतिवेदन तयार गर्दा नेपाल सरकारले निर्धारण गरेको मापदण्ड एवं गुणस्तर कायम हुनेगरी तोकिएको ढाँचामा तयार गर्नु पर्नेछ।

  २. उपदफा १ बमोजिमको मापदण्ड वा गुणस्तर विपरित वा त्यस्तो मापदण्ड पालना नगरी प्रतिवेदन पेश भएमा त्यस्तो प्रतिवेदन तयार गर्ने **परामर्शदाताले बढीमा ५ वर्ष सम्म** वातावरणीय अध्ययन प्रतिवेदन तयार गर्न पाउने छैन।
- **रणनीतिक वातावरणीय विश्लेषण (दफा ९):** नेपाल सरकारले नेपाल राजपत्रमा सूचना प्रकाशन गरी तोकेका नीति, कार्यक्रम वा आयोजना कार्यन्वयन गर्नु अघि त्यस्तो नीति, कार्यक्रम वा आयोजनाको सम्बन्धमा रणनीतिक वातावरणीय विश्लेषण गर्नु पर्नेछ।
- **वातावरणीय व्यस्थापन योजना (दफा १०):**
- **पुरक वातावरणीय प्रभाव मूल्यांकन प्रतिवेदन (दफा ११):** एक पटक स्वीकृत भैसकेको वातावरणीय प्रभाव मूल्यांकन सम्बन्धी प्रस्तावमा आँशिक रुपमा भौतिक पूर्वाधार, डिजाइन वा स्वरूप परिवर्तन गर्न, संरचना स्थानान्तरण वा फेरबदल गर्न, वन क्षेत्र थप गर्न वा आयोजनाको क्षमता वृद्धि गर्नु परेमा त्यस्तो कार्य वातावरणमा प्रतिकुल प्रभाव पर्ने वा नपर्ने, त्यस्तो प्रभावलाई कुनै उपायद्वारा निराकरण वा न्यूनीकरण गर्न सकिने वा नसकिने सम्बन्धमा यकिन गर्न प्रस्तावकले पु.वा.प्र.मू. गर्नु पर्नेछ।

वातावरण संरक्षण ऐन,२०७६

- **वातावरणीय परीक्षण (दफा १२): *मन्त्रालय वा तोकिएको निकायले*** यस ऐन बमोजिम वातावरणीय प्रभाव मूल्यांकन गर्नुपर्ने प्रस्तावको कार्यान्यिवन सुरु गरी सेवा वा वस्तु उत्पादन वा वितरण सुरु गरेको दुई वर्ष भुक्तान भएको मितिले ६ महिना भित्र त्यस्तो प्रस्तावको कार्यन्वयनबाट वातावणमा परेको प्रतिकुल प्रभाव, त्यस्तो प्रभावलाई कम गर्न अपनाएको उपाय तथा त्यस्तो उपायको प्रभावकारिता र न्यूनीकरण हुन नसकेको वा आकलन नै नभएको प्रतिकुल प्रभाव उत्पन्न भएकोमा सो समेतको विश्लेषण गरी वातावरणीय परीक्षण प्रतिवेदन अध्यावधिक राख्नु पर्नेछ।
- **जरिवाना(दफा ३५):-**
  - संक्षिप्त वातावरणीय अध्ययन– पाँच लाख रुपैया सम्म।
  - प्रारम्भिक वातावरणीय परीक्षण– दश लाख रुपैया सम्म।
  - वातावरणीय प्रभाव मूल्यांकन– पचास लाख रुपैया सम्म। ***(स्वीकृत नगरे वा स्वीकृत प्रतिवेदनको विपरित हुनेगरी)***
  - सम्बन्धित निकायले प्रतिवेदन स्वीकृत गराउन वा प्रतिवेदन विपरित कार्य भएमा सुधार गर्न आदेश दिनेछ
  - त्यस्तो आदेश पालना गर्नु सम्बन्धित व्यक्ति वा संस्थाको कर्तब्य हुनेछ।
  - यसरी दिएको आदेश बमोजिम कार्य नभएमा सम्बन्धित निकायले तेब्बर जरिवाना गर्ने छ।

वातावरण संरक्षण ऐन,२०७६

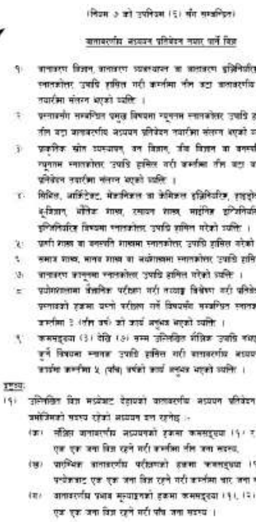
- **निवेदन दिन सक्ने (दफा ३६):-** कसैले यस ऐन विपरित स.वा.अ., प्रा. वा.प. वा वा.प्र.मू. प्रतिवेदन स्वीकृत नगराई वा स्वीकृत प्रतिवेदन विपरित हुनेगरी प्रस्ताव कार्यन्यवन गरेमा वा गर्न लागेमा **सम्बन्धित निकाय वा अधिकारी** समक्ष निवेदन दिन सक्नेछ।
  - हानी नोक्सानी भएको खण्डमा प्रस्तावकबाट पीडित व्यक्ति, संस्था वा स्थानीय समुदायलाई मनाशिव क्षतिपूर्ति भराई दिनुपर्ने छ।

- **अनुगमन तथा निरीक्षण गर्ने (दफा ३९):-** यस ऐन वा ऐन अन्तर्गत बनेको नियम निर्देशिका, कार्यविधि वा मापदण्डको कार्यान्वयन भए नभएको सम्बन्धमा मन्त्रालय (**वन तथा वातावरण मन्त्रालय**) वा विभाग (**वातावरण विभाग**) ले अनुगमन तथा निरीक्षण गर्नेछ।

वातावरण संरक्षण नियमावली,२०७७

- **वातावरणीय अध्ययन गर्नु पर्ने (नियम ३):**
  - ***अनुसूची-१*** संक्षिप्त वातावरणीय अध्ययन {क-झ}
  - ***अनुसूची-२*** प्रारम्भिक वातावरणीय परीक्षण {क-ठ}
  - ***अनुसूची-३*** वातावरणीय प्रभाव मूल्याङ्कन (क-ठ}
- **क्षेत्र निर्धारण गर्नु पर्ने (नियम ४):** वातावरणीय प्रभाव मूल्याङ्कन **(अनुसूची ४ र ५)**
- **कार्यसूची तयार गर्नु पर्ने (नियम ५):**
  - (क) संक्षिप्त वातावरणीय अध्ययनसँग सम्बन्धित कार्यसूची **अनुसूची-६**
  - (ख) प्रारम्भिक वातावरणीय परीक्षणसँग सम्बन्धित कार्यसूची **अनुसूची-७**
  - (ग) वातावरणीय प्रभाव मूल्याङ्कनसँग सम्बन्धित कार्यसूची **अनुसूची-८**
- **सार्वजनिक सुनुवाई गर्नु पर्ने (नियम ६):**
- **वातावरणीय अध्ययन प्रतिवेदन तयार गर्ने(नियम ७):**
  - (क) संक्षिप्त वातावरणीय अध्ययन प्रतिवेदन **अनुसूची-१०**
  - (ख) प्रारम्भिक वातावरणीय परीक्षण प्रतिवेदन **अनुसूची-११**
  - (ग) वातावरणीय प्रभाव मूल्याङ्कन प्रतिवेदन **अनुसूची-१२**

  *(**अनुसूची-१३** बमोजिमका विज्ञ मार्फत तयार गर्नु पर्नेछ)*



वातावरण संरक्षण नियमावली,२०७७

- **वातावरणीय अध्ययन प्रतिवेदन स्वीकृतिको लागि पेश गर्नु पर्ने (नियम ८):**
  - अनुसूची-१४ बमोजिमको ढाँचामा सम्बन्धित स्थानीय तह र सम्बन्धित विषयगत कार्यालयको सिफारिस समेत समावेश गर्नु पर्नेछ।
- **वातावरणीय अध्ययन प्रतिवेदन स्वीकृत गर्ने (नियम ९):**

- **रणनीतिक वातावरणीय विश्लेषण (नियम १०):**
- **पूरक वातावरणीय प्रभाव मूल्याङ्कन सम्बन्धी व्यवस्था(नियम ११):**

पूरक वातावरणीय प्रभाव मूल्यांकन (दफा-११)
कुनै आयोजनाको भौतिक पूर्वाधार डिजाइन वा स्वरुपमा केहि परिवर्तन गर्नु परेमा, संरचना फेरबदल वा स्थान्तरण गर्नु परेमा, बनक्षेत्र थप गर्नु परेमा वा आयोजनाको क्षमता वृद्धि गर्नु परेमा।

स.वा.अ. र प्रा.वा.प.परिमार्जन गर्न सकिने (नियम-१२)
कुनै आयोजनाको केही भौतिक पूर्वाधार, डिजाइन, क्षमता अभिवृद्धि वा स्वरूप परिमार्जन गर्नु पर्ने वा संरचना स्थान्तरण गर्नु पर्ने भएमा वा आयोजनाको क्षमता घटेमा वा रुख संख्या थपघट गर्नु पर्ने भएमा।

वातावरणीय व्यवस्थापन योजना (दफा-१० र नियम.११-६)
एक पटक वातावरणीय प्रभाव मूल्यांकन स्वीकृत भैसकेको आयोजनामा केही भौतिक पूर्वाधार, डिजाइन वा स्वरूप परिमार्जन गर्नु पर्ने वा संरचना स्थान्तरण वा फेरवदल गर्नु पर्ने भएता पनि पूरक वातावरणीय प्रभाव मूल्यांकन गर्नु पर्ने अवस्था नदेखिएमा वा आयोजनाको क्षमता घटेमा वा रुख कटान संख्यामा थपघट गर्नु पर्ने भएमा सम्बन्धित निकायले वातावरणीय प्रभाव मूल्यांकन प्रतिवेदनमा रहेको वातावरणीय व्यवस्थापन योजना परिमार्जन गर्न स्विकृती दिन सक्नेछ।

वातावरण संरक्षण नियमावली,२०७७

- **संक्षिप्त वातावरणीय अध्ययन प्रतिवेदन वा प्रारम्भिक वातावरणीय परीक्षण प्रतिवेदन परिमार्जन गर्न सकिने(नियम १२):**
- **क्षर्तिपूर्ति भराउने (नियम ४१):** ऐन को दफा ३५ बमोजिम
- **अनुगमन तथा निरीक्षण(नियम ८):**
  - (१) **प्रस्तावकले** प्रस्तावको निर्माण तथा सञ्चालन गर्ने चरणमा सोबाट वातावरणमा परेको प्रभावको विषयमा **प्रत्येक छ महिनामा स्वःअनुगमन गरी** सोको प्रतिवेदन सम्बन्धित निकाय वा विभागमा पेश गर्नु पर्नेछ।
  - (२) ऐनको दफा ३९ बमोजिम **मन्त्रालय वा विभागले** कुनै आयोजनाको अनुगमन तथा निरीक्षण गर्दा प्रस्ताव स्वीकृत हुँदाका बखतको वातावरणीय अध्ययन प्रतिवेदनमा उल्लिखित सीमाभन्दा बढी प्रभाव परेको देखिएमा त्यस्ता प्रभाव हटाउन वा हटाउने उपाय अवलम्बन गर्न सो आयोजनाको प्रस्तावकलाई निर्देशन दिनेछ र त्यस्तो निर्देशनको पालना गर्नु सम्बन्धित प्रस्तावकको कर्तव्य हुनेछ।

वातावरण संरक्षण नियमावली,२०७७

- नेपाली भाषामा प्रतिवेदन गर्नु पर्ने *(अन्तर्राष्ट्रिय दाताको लागि अंग्रेजीमा)*
- सार्वजनिक सूचना, सिफारिस पत्र, क्षेत्र निर्धारण प्रतिवेदन, कार्यसूची प्रतिवेदन, सबै अन्य प्रतिवेदनका ढाँचा संलग्न
- समय सीमा निर्धारण:
  - कार्यसूची/ र क्षेत्र निर्धारण: १५ दिन भित्र
  - स.वा.अ.र.प्रा.वा.प.: १५ दिन भित्र
  - वा.प्र.मू: ३५ दिन भित्र
  - कार्यसूची र क्षेत्र निर्धारण भएको २ बर्ष भित्र वातावरणीय अध्ययन गरीसक्नु पर्ने
  - कुनै कारणबस २ बर्ष भित्र गर्न नसकिए थप १ वर्ष म्याद थप हुनसक्ने
  - एकिकृत प्रतिवेदन समेत तयार गर्न सकिने: एक भन्दा वढी सरोकार हुंदा
- प्रतिवेदन स्वीकृत भएको ३ वर्ष भित्र प्रस्ताव कार्यान्वन सुरु गर्नु पर्ने: निर्धारित समयमा कार्य सुरु गर्न नसकिए सम्बन्धित निकायले बढीमा २ वर्ष अवधि थप गर्न सक्ने

12/6/2020

अनुसूची १ : BES ,अनुसूची २ : IEE ,अनुसूची ३ : EIA ,अनुसूची ४ : SEA

For IEE

i) Construction of bridge more than 250 m
ii) All New Road construction up to 25 km except local road.
iii) Construction of 5~50 km long ropeways,
iv) Construction of 1~5 km long cable car routes,
v) Construction of tunnel up to 3 km
vi) Fly over and monorail route
vii) 10 to 50 km National Highway for upgrading/width increasing /reconstruction etc.
viii) National/internal water way route

Proposal requiring ELA: All which are more than the limit of IEE stated above.

For

Energy, Water Resources and Irrigation

BES: Up to 100 hector lift irrigation, up to 66 KV Electricity Distribution Line project using forest and Water and Climate Measurement Center or Climate Radar work need forest.

IEE

जलस्रोत र ऊर्जा क्षेत्र :

(१) विद्युत प्रसारण लाईन र सबस्टेशन निर्माण अन्तर्गत:-

(क) १३२ के.भी. वा सोभन्दा बढी क्षमताको विद्युत प्रसारण लाइन निर्माण गर्ने,

(ख) विद्यमान २२० के.भी. वा सोभन्दा बढी क्षमताको विद्युत प्रसारण लाइनबाट टयाप गरी नयाँ आउटडोर सबस्टेशन निर्माण गर्ने ।

(२) विद्युत उत्पादन अन्तर्गत:-

(क) १ देखि ५० मेगावाट क्षमतासम्मको जलविद्युत उत्पादन आयोजना निर्माण गर्ने,

(ख) १ देखि ५ मेगावाट क्षमता सम्मको खनिज तेल वा ग्यासबाट विद्युत उत्पादन आयोजना सञ्चालन गर्ने,

(ग) १५०० घनमिटर भन्दा बढी क्षमताको बायोग्यास प्लान्ट निर्माण गर्ने ।

(४) सिँचाईको पुनरुत्थान प्रणाली अन्तर्गत: विद्यमान प्रणाली भित्रका सिँचाइ आयोजनामा नयाँ हेडवर्क्स निर्माण वा मूल नहर परिवर्तन हुने कुनै पनि पुनरुत्थान आयोजना सञ्चालन गर्ने,

(५) २५ देखि १०० जनसम्म स्थायी बसोबास भएको जनसङ्ख्या विस्थापित गर्ने कुनै पनि जल विकास कार्य गर्ने,

(६) १० किलोमिटर भन्दा बढी लम्बाईको नदी नियन्त्रणको कार्य गर्ने,

(७) नवीकरणीय ऊर्जा क्षेत्र:

(क) १ देखि १० मेगावाट क्षमतासम्मको सौर्य ऊर्जाबाट विद्युत उत्पादन आयोजना सञ्चालन गर्ने,

(ख) १ देखि १० मेगावाट क्षमतासम्मको वायु ऊर्जाबाट विद्युत उत्पादन आयोजना सञ्चालन गर्ने,

FOR EIA

जलस्रोत र ऊर्जा क्षेत्र :

(१) विद्युत उत्पादन अन्तर्गतः-

(क) ५० मेगावाटभन्दा बढी क्षमताको जलवि उत्पादन आयोजना निर्माण गर्ने,

(ख) १ मेगावाटभन्दा बढी क्षमताको कोइला आणविक विद्युत उत्पादन आयोज सञ्चालन गर्ने,

(ग) ५ मेगावाटभन्दा बढी क्षमताको खनिज तेल ग्यासबाट विद्युत उत्पादन आयोजना सञ्चा गर्ने ।

(२) सिँचाइको नयाँ प्रणाली अन्तर्गतः-

(क) तराई वा भित्री मधेशमा २,००० हेक्टरभ बढीको क्षेत्र सिँचाइ गर्ने,

(ख) पहाडी उपत्यका र टारमा ५०० हेक्टरभ बढीको क्षेत्र सिँचाइ गर्ने,

(ग) पहाडी भिरालो पाखा वा पर्वतीय क्षेत्रमा हेक्टर भन्दा बढी क्षेत्र सिँचाइ गर्ने ।

(३) १०० जनाभन्दा बढी स्थायी बसोबास जनसङ्ख्या विस्थापित गर्ने कुनै पनि जल विकास कार्य गर्ने,

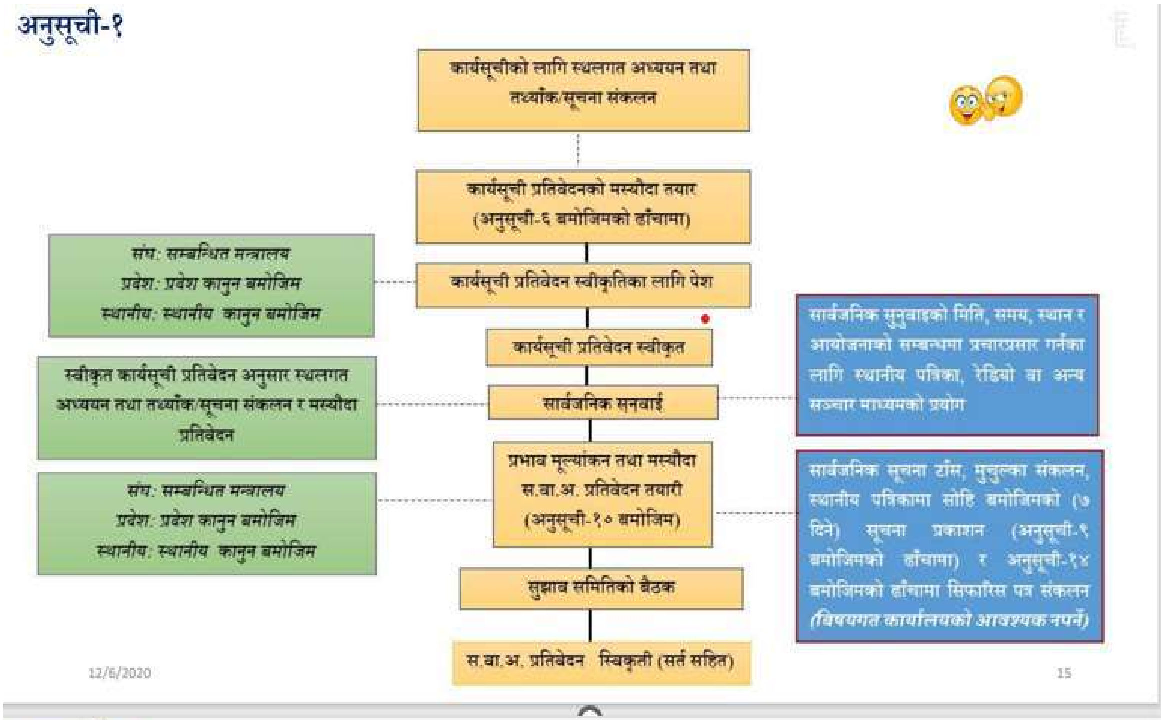
(४) बहुउद्देश्यीय जलाशयको निर्माण गर्ने,

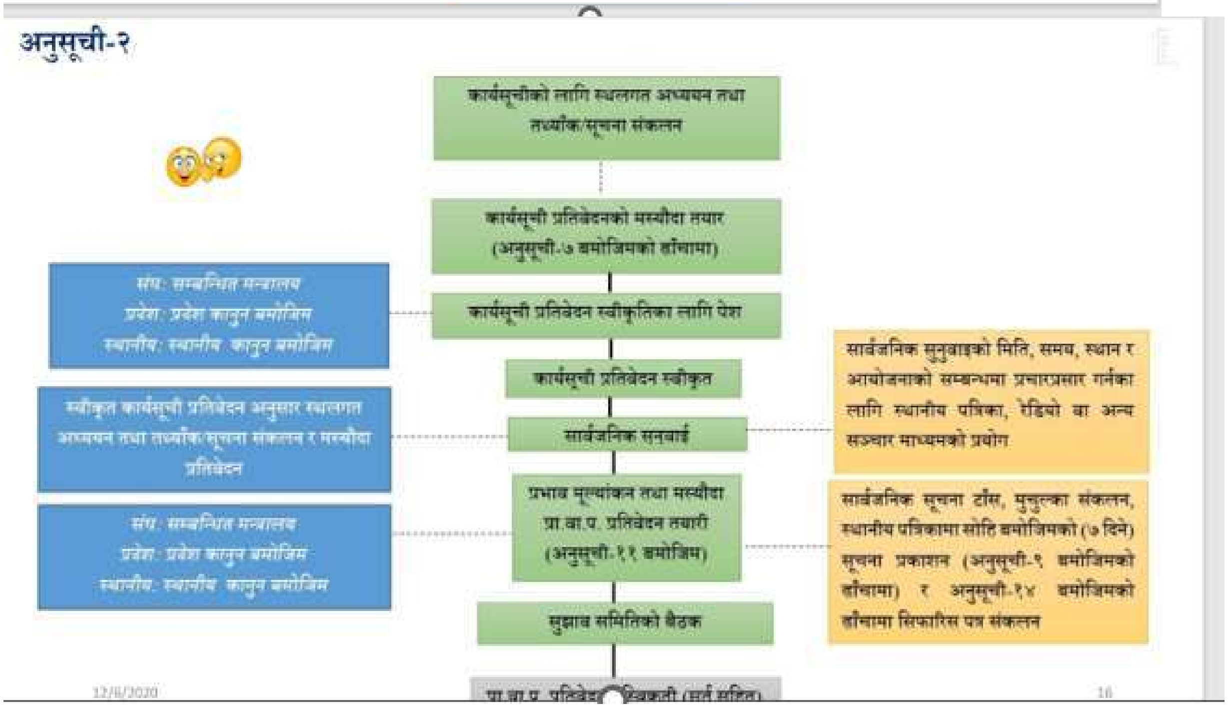
(५) एउटा जलाधार क्षेत्रबाट अर्को जलाधार क्षेत्रमा फर्काई (इन्टर बेसिन वाटर ट्रान्सफर) उपयोग

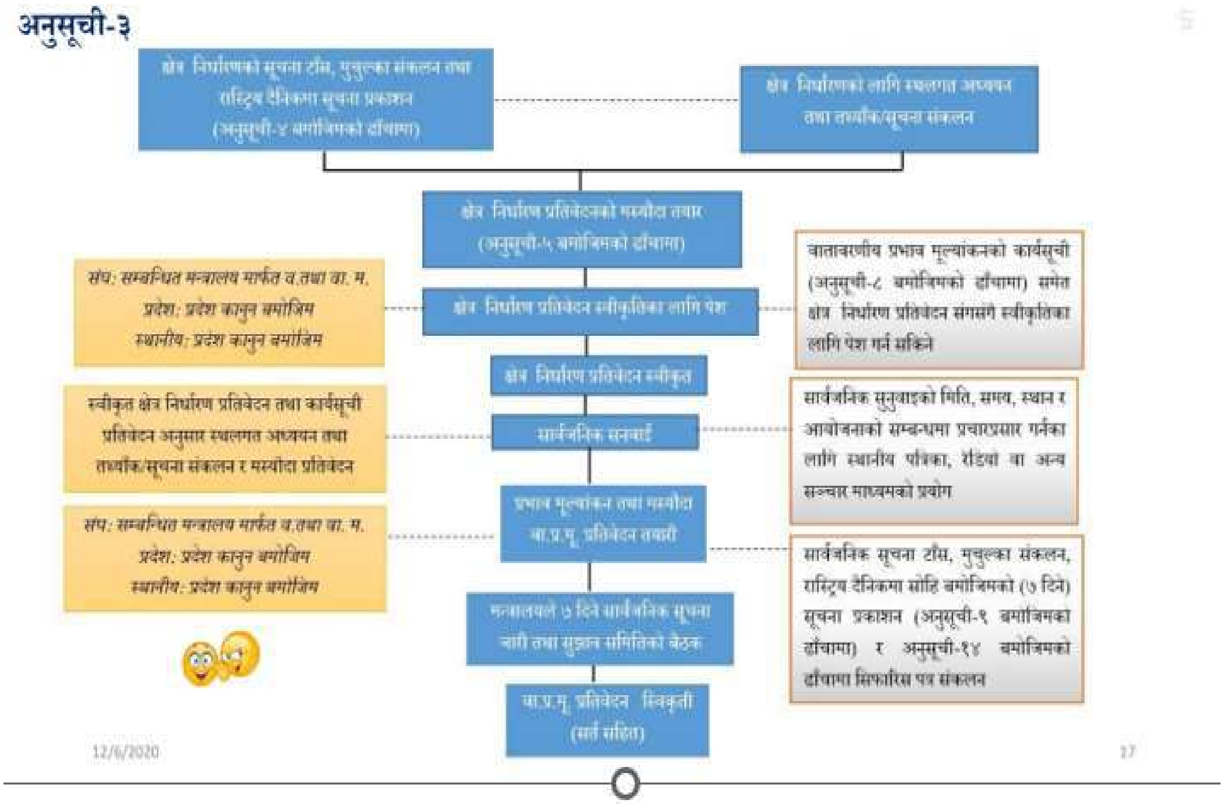
(६) नवीकरणीय ऊर्जा क्षेत्र :-

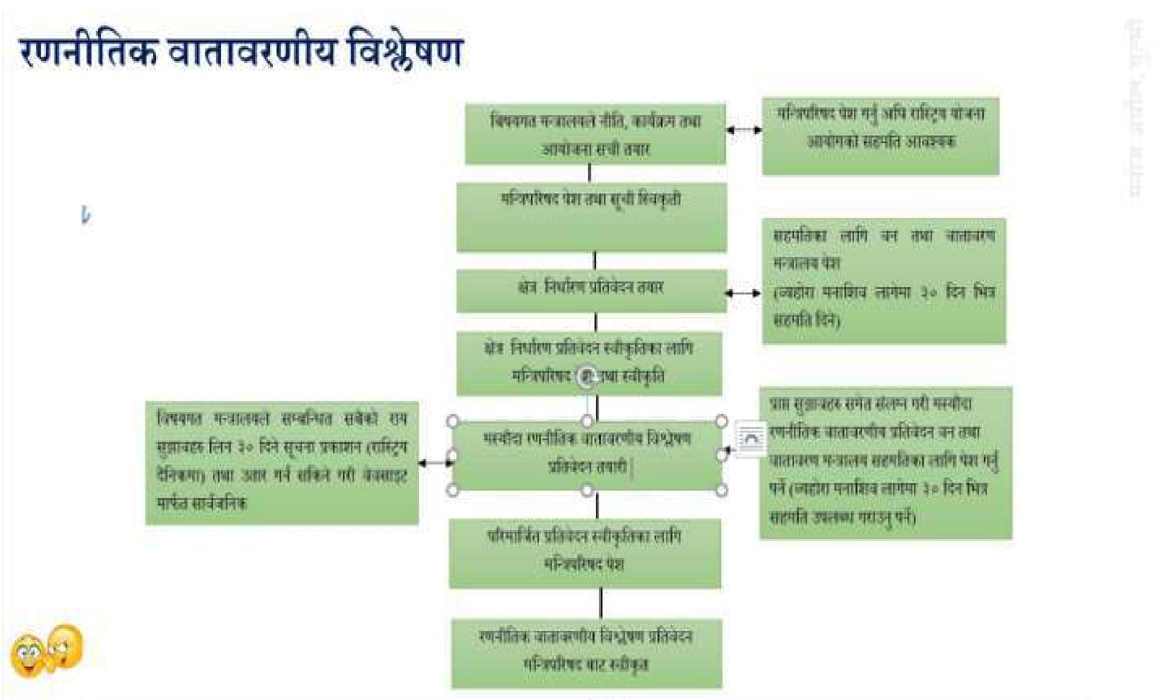
(क) १० मेगावाट भन्दा बढी क्षमताको सौर्य ऊ विद्युत उत्पादन आयोजना सञ्चालन गर्ने,

(ख) १० मेगावाट भन्दा बढी क्षमताको वायु ऊ


अनुसूची-१
कार्यसूचीको लागि स्थलगत अध्ययन तथा तथ्यांक/सूचना संकलन
कार्यसूची प्रतिवेदनको मस्यौदा तयार (अनुसूची-६ बमोजिमको ढाँचामा)
संघ: सम्बन्धित मन्त्रालय
प्रदेश: प्रदेश कानुन बमोजिम
स्थानीय: स्थानीय कानुन बमोजिम
कार्यसूची प्रतिवेदन स्वीकृतिका लागि पेश
कार्यसूची प्रतिवेदन स्वीकृत
सार्वजनिक सुनुवाइको मिति, समय, स्थान र आयोजनाको सम्बन्धमा प्रचारप्रसार गर्नका लागि स्थानीय पत्रिका, रेडियो वा अन्य सञ्चार माध्यमको प्रयोग
स्वीकृत कार्यसूची प्रतिवेदन अनुसार स्थलगत अध्ययन तथा तथ्यांक/सूचना संकलन र मस्यौदा प्रतिवेदन
सार्वजनिक सनवाई
प्रभाव मूल्यांकन तथा मस्यौदा स.वा.अ. प्रतिवेदन तयारी (अनुसूची-१० बमोजिम)
संघ: सम्बन्धित मन्त्रालय
प्रदेश: प्रदेश कानुन बमोजिम
स्थानीय: स्थानीय कानुन बमोजिम
सार्वजनिक सूचना टाँस, मुचुल्का संकलन, स्थानीय पत्रिकामा सोहि बमोजिमको (७ दिने) सूचना प्रकाशन (अनुसूची-९ बमोजिमको ढाँचामा) र अनुसूची-१४ बमोजिमको ढाँचामा सिफारिस पत्र संकलन (विषयगत कार्यालयको आवश्यक नपर्ने)
सुझाव समितिको बैठक
स.वा.अ. प्रतिवेदन स्विकृती (सर्त सहित)
12/6/2020
15


अनुसूची-२
कार्यसूचीको लागि स्थलगत अध्ययन तथा तथ्यांक/सूचना संकलन
कार्यसूची प्रतिवेदनको मस्यौदा तयार (अनुसूची-७ बमोजिमको ढाँचामा)
संघ: सम्बन्धित मन्त्रालय
प्रदेश: प्रदेश कानुन बमोजिम
स्थानीय: स्थानीय कानुन बमोजिम
कार्यसूची प्रतिवेदन स्वीकृतिका लागि पेश
कार्यसूची प्रतिवेदन स्वीकृत
सार्वजनिक सुनुवाइको मिति, समय, स्थान र आयोजनाको सम्बन्धमा प्रचारप्रसार गर्नका लागि स्थानीय पत्रिका, रेडियो वा अन्य सञ्चार माध्यमको प्रयोग
स्वीकृत कार्यसूची प्रतिवेदन अनुसार स्थलगत अध्ययन तथा तथ्यांक/सूचना संकलन र मस्यौदा प्रतिवेदन
सार्वजनिक सनवाई
प्रभाव मूल्यांकन तथा मस्यौदा प्रा.वा.प. प्रतिवेदन तयारी (अनुसूची-११ बमोजिम)
संघ: सम्बन्धित मन्त्रालय
प्रदेश: प्रदेश कानुन बमोजिम
स्थानीय: स्थानीय कानुन बमोजिम
सार्वजनिक सूचना टाँस, मुचुल्का संकलन, स्थानीय पत्रिकामा सोहि बमोजिमको (७ दिने) सूचना प्रकाशन (अनुसूची-९ बमोजिमको ढाँचामा) र अनुसूची-१४ बमोजिमको ढाँचामा सिफारिस पत्र संकलन
सुझाव समितिको बैठक
12/6/2020
16





वातावरणीय अध्ययन/मूल्यांकनको उदेश्यहरु/महत्वहरु

- प्रस्ताव भएका र हुने परियोजनाहरु वातावरणीय ,सामाजिक, आर्थिक, सांस्कृतिक दृष्टिकोणबाट ठिक छ छैन यकिन गरी दिगो बिकासमा टेवा पुर्याउन
- निर्णयकर्ताहरुलाई योजनामा पर्न सक्ने वातावरणीय तथा सामाजिक जोखिम बारे जानकारी गराउन
- निर्णय प्रक्रियामा निर्णयकर्ता, सम्बन्धित सरोकारवालाको सक्रिय सहभागिता र पारदर्शिता बढाउन
- नेपालको संविधानको धारा ३० ले प्रत्येक नागरिकलाई स्वच्छ वातावारंमा बाँच्न पाउने मौलिक हकको को संरक्षण गर्न

- दिगो विकासको अवधारणा अनुरुप वर्तमान र भावीपुस्ताको समन्यायका लागि वातावरणीय स्रोतमाथि न्यायोचित पहुँच,वितरण र तिनको बुद्धिमतापूर्ण उपयोगको प्रत्याभूति गर्न
- विकास आयोजना हरुको सबै चरणमा वातावरणीय पक्षलाई आन्तारिकिकरण गर्न
- विकास आयोजनाबाट सृजित प्रतिकूल वातावरणीय प्रभावमा परेको समुदायलाई न्यायोचित क्षतिपूर्तिको व्यवस्था गर्न
- भौतिक पूर्वाधारको निर्माण गर्दा वातावरणमैत्री संरचना निर्माण गर्न,प्रतिकूल प्रभाव निराकरण गर्न र अनुकुल प्रभाव वृद्धि गर्न साथै कुनै उपाय द्वारा जोखिमको न्यूनीकरण गर्न नसक्ने परियोजनालाई कार्यान्वयनमा रोक लगाउन
- परियोजनाबाट सामाजिक तथा आर्थिक अवसरहरु सिर्जना गर्दै सरोकारवालाको निर्णय प्रक्रियामा सहभागी गराउने र उनीहरुको क्षमता विकास गर्न
- Payment for Ecosystem and Polluter pay Principle लाई आत्मसात गर्न
- महत्वपूर्ण व्यवस्थापकीय tool) costlessthan %0.1 ofprojectcost (जसले परियोजनाको दिगोपनमा सहयोग गर्ने
- विकास प्रक्रियामा हुने अवरोध हटाई सम्बन्धित पक्षमा अपनत्व ग्रहण गराउन आदि

BESorIEE को प्रतिवेदनमा हुने वातावरणीय योजनाको खाँका वा EIA मा हुने EnvironmentalManagementPlan) EMP (को खाँका

विषयगत क्षेत्र	सकारात्मक प्रभावको बढोत्तरी	के के गर्ने	कहाँ गर्ने	कसरी गर्ने	कहिले गर्ने	कसले गर्ने	अनुमानित स्रोतसाधन जनशक्ति	MandE	कैफियत
भौतिक									
जैविक									
समाजिक									
सांस्कृतिक									
अन्य									
नकरात्मक प्रभावको न्यूनीकरण गर्ने क्रियाकलाप									
भौतिक									
....									
अन्य									

वातावरणमा पर्ने प्रतिकुल प्रभावलाई हटाउने वा न्यून गर्ने उपायहरु

- क्षतिपूर्तिका उपाय)CompensatoryMeasures :(जस्तै क्षतिग्रस्त प्राकृतिक स्रोतको पुनर्स्थापना ,हटाउने बस्तीको पुनर्बास,प्रभावितलाई क्षतिपूर्ति आदि
- सुधारात्मक उपाय)CorrectiveMeasures :(जस्तै प्रदुषण नियन्त्रण उपकरणको जडान, प्रदुषित पानीको उपचार गर्ने संयन्त्र निर्माण, बाँध तथा तटबन्धमा FishLadder निर्माण आदि
- प्रतिरोधात्मक उपाय)PreventiveMeasures :(स्वास्थ शिक्षा कार्यक्रमको कार्यान्वयन ,जनचेतनाको कार्यक्रम थालनी, समाजिक सहयोग जस्ता कार्य

माथि उल्लेखित संरक्षणक उपायहरु मौलिक, जैविक, समाजिक आर्थिक तथा सांस्कृतिक वातावरण क क्षेत्रमा प्रस्तावको निर्माण तथा संचालन अवस्थाको लागि भनी बर्गिकरण गर्नुपर्ने

अनुगमनका प्रकार

- प्रारम्भिक अवस्थाको अनुगमन)BaseLineMonitoring :(प्रस्ताव कार्यान्वयन गर्नुभन्दा अगावै सो क्षेत्रको वरपरका आधारभूत वातावरणीय पक्षको survey|
- प्रभाव अनुगमन)ImpactMonitoring :(प्रस्ताव कार्यान्वयनबाट भएका वातावरणीय परिवर्तन पत्ता लगाउन आयोजन निर्माण र संचालनका क्रममा त्यस क्षेत्रको जनस्वास्थ लगायत पर्यावरणीय /सामाजिक/आर्थिक अवस्थाको सूचकको मूल्यांकन गर्नुपर्ने कार्य |
- नियमपालन अनुगमन)ComplianceMonitoring :(वातावरण संरक्षण सम्बन्धि निर्धारित मापदण्ड पालना गरेको छ भन्ने कुरा सुनिश्चित गर्ने वातावरणीय गुणस्तरका विशेष सूचक वा प्रदुशंको अवस्था बारेमा आवधिक वा लगाताररुपमा गरिने अनुगमन |

Project Cycle and Relevant EIA Activities

StagesinProjectCycle	**RecommendedEIAActivities**
1. **PROJECTCONCEPT/IDENTIFICATION** • identificationofproject • detaildesignnotavailable • basicnatureofprojectknown	Screening - to decide what sort of environmentstudy is necessary; full EIA, IEE Approve, andReject.(sensitivearea, threshold, size criteria) • choose environmentalviable alternatives • indicationsofkey impacts for further study.
2. PRE-FEASIBILITY • preliminary comparisonofalternatives andselectionofviableonesforfuture investigation/ evaluation	Scoping- identification of issues/impacts for investigation,formulationofTOR for EIA, decision ismade on, • impactstobeinvestigated • EIAworkschedules • consultationtobe undertaken • methods/techniquestobeused • form/contentofEIAreport
s. **FEASIBILITY**STUDY • economicandtechnical comparisonof the alternatives • detail engineering design and constructiondetails of theselected alternative	MajorityofEIAstudywork • identification, prediction and assessmentof impacts andtheir significance • identificationanddesign of mitigation measures • developmentofmonitoringplan
4. Project Appraisalanddecisionbeforeimplementationofaproject,itisreviewedinlight ofthefeasibilitystudyandEIAstudyfindings.Decisionistobemadeastowhetherthe projectbeimplemented (go-ahead).	
6. PROJECTIMPLEMENTATIONPLAN • consultant • contractor,financeand reportingand evaluation	Preparation**ofEnvironmental** ManagementActionPlan(EMAP)
s. ENGTNEERTNGDESIGN, COST ESTIMATE,CONTACT**PACKAGEAND**BIDDING DOCUMENT	IntegrationEMAPrequirementin (i)design(ii)costestimate(iii)contractor package(iv)BOO(v)tenderdocument
MONITORINGANDAUDITING	EMAPCompliance

EMP मा भएको प्राबधान लागू भए/नभएको अवस्थाको विश्लेषण

- शतप्रतिशत लागू वा प्रयोग नभएपनि आयोजना पहिचानको बेला नै MTEF ,नतिजामूलक खाँकाको प्रावधान तयार गरिएको तर field मा लागू गर्न कठिन ।
- जव सम्म नतिजामूलक अनुगमन हुदैन तब सम्म Implementation गर्न कठिन ।
- जरिवाना वा कारवाहीक प्रावधानहरु पनि अनुगमन कै आधारमा हुने तर अनुगमनको अवस्था नाजुक भएकोले वास्तविकरुपमा कार्यान्वयन पक्ष कमजोर रहेको तर केहि विषयहरु जस्तै
    - ढुंगा गिट्टी उत्खनन्,
    - रुख काट्ने,
    - बन क्षेत्रको जग्गा प्रयोग,
    - प्रभावित समुहलाई रोजगारमूलक तालिम
    - पुनः रुख बिरुवा रोप्नुपर्ने जस्ता प्रावधानहरु लागू भैरहेको
- ठेक्का शर्तहरुमा EMP का प्रावधानहरु राखेर लागू गर्ने प्रयास
- BoQ मा वातावरणीय बिषयबस्तुलाई समाबेश गरी लागू गरिएको /BioEngineering लाई प्राथमिकताक साथ लागू गरेको आदि

तसर्थ EMP क कतिपय प्रावधानहरु कार्यान्वयन हुदै आएको छ भने कतिपय अझै प्रभावकारी रुपमा कार्यान्वयन गराउन नसकेको वर्तमान अवस्थामा यसका कमीकमजोरीलाई निराकरण गर्दै SDG लक्ष्यलाई हासिल गर्नुपर्ने देखिन्छ ।

EPR and EPR विकास निर्माणमा Hurdles गरेको छ त ?सुधारात्मक प्रयत्नहरु के के हुन् सक्छन ?

- EPRandEPR विकास निर्माणमा आफैमा hurdles होइन तर ऐन र नियमावलीमा भएका कतिपय प्राबधान हरु प्रशासनिक र बैज्ञानिक बस्तुगत document भएपनि कार्यान्वयनक चरणमा त्यसका प्रक्रियाहरु जस्ता सूचना प्रकाशन, मुचुल्का उठाउने, राय सुझाव लिने जस्ता काम tedious भएको
- बन तथा वातावरण सम्बन्धि निकायले EmpireBuilding युक्त मनोभावना ले व्यवहारिक पक्ष् कमजोर देखिएको
- SDG मा वनक्षेत्र ४८.२ प्रतिशत बनले ढाकेको क्षेत्र पुर्याउने उल्लेख भएकोमा सार्वजनिक निर्माण गर्दा मुआब्जा नदिने/जग्गा अधिग्रहण नगरी निर्माण गर्दा सार्वजनिक निकायले सकेसम्म व्यक्तिको सम्पति/जग्गा छल्ने मनसायले बनक्षेत्र भित्र लाने खोज्ने तर बन क्षेत्र प्रयोगको लागि IEE/EIA ,रुख कटानको लागि मन्त्रिपरिषद्को सहमति लिनुपर्ने, पुनः 1:25Compensatory रुख रोप्नुपर्ने,सो को लागि जग्गाको रकम, हुर्काउन लाग्ने रकम सबै परियोजना कार्यालयले नै व्यहोर्नु पर्ने जस्ता कार्य साथै बनमन्त्रालयले आफ्नो मातहतका कार्यालयमा प्रतिवेदन पुनः पठाई सिफारिस माग्ने, सिफारिश नदिए प्रोजेक्ट नै धरापमा पर्ने जस्ता कठिनाई ले हाम्रो वातावरणीय मुद्दा GreenTapism त होइन भन्ने भान हुने गर्दछ तर SDG को एक पिलर वातावरण नै भएकोले यसको अभावमा SDG को लक्ष्य हासिल हुन सक्दैन त्यसैले वातावारणमैत्री पूर्वाधार र दिगो बिकासको अवधारणालाइ आत्मसात गर्न जमीन,

जमिनमुनि र जमिनमाथीको वातावरण र जीवनचक्रलाई व्यवस्थित बनाउन EPR/EPA को भूमिका महत्वपूर्ण भएकोले प्रक्रियागत सहजीकरण मार्फत यसको प्रयोग गर्नुपर्ने अनिवार्यता रहेको छ ।

EPA र EPR मा सुधार गर्नुपर्ने केहि बिषयबस्तु

- अनुगमन प्रस्तावकले गर्ने :सैदान्तिक रुपमा बिकसित मुलुकमा यसको प्रयोग ठोस रुपमा भए पनि हाम्रो जस्तो विकाशशिल देशमा पूर्वाधारमा लगानी बढाउनु पर्ने र निर्माण क्षेत्रमा तिब्रता दिनुपर्ने अवस्थामा स्रोतसाधनको कमिले प्रभावकारी रुपमा लागू गर्न कठिनाई ।
- GreenTapism मा सुधार र अन्तर निकाय समन्वयमा जोड
- प्रक्रियागत कतिपय प्राबधानहरुमा सुधार गर्नुपर्ने जस्तै EIA मा ४ पटक सूचना /सार्वाजनिक सुनुवाई
- राष्ट्रिय प्राथमिकता प्राप्त आयोजनाको लागि राष्ट्रिय बनक्षेत्र् प्रयोग गर्ने मापदण्ड सहितको कार्यविधिमा IEE/EIA स्वीकृत गर्नभन्दा अगाडी बन मन्त्रलयको सहमति लिनुपर्ने जुन irrelevant छ किनकि सामुदायिक बन क्षेत्र/राष्ट्रिय बन हरु प्रदेशमा गएको अवस्थामा झन्झटिलो प्रक्रिया र अन्त्यनै लामो प्रक्रिया ।
- राय सुझाव सार्वजनिक सूचना बाट माग गरिसकेपछि बन मन्त्रालयले कुनै particularproject को लागि छुट्टै आफै राय सुझाव दिन सक्दैन र उसले आफ्नो मतहातको कार्यालयमा पठाउदा लामो समय लाग्ने वा आयोजना नै ढिला वा implementation गर्न नमिल्ने सुझाव आउन सक्ने र आयोजना Cancel हुने ।
- कतिपय अवस्थामा जस्तै designandBuiltBridges हरुमा पुल निर्माण सम्पन्न भै सकेपछि पनि वातावरणीय अध्ययन नसकेको अवस्थालाइ सुधार गरी कुन चरणमा के काम गर्ने हो परियोजनाको कार्य संग Tieup हुनुपर्ने ।
- स्थानीय तहमा IEE/BES मा आफै प्रस्तावक, आफै Monitoring ,approval र implementation कर्ता अनि ढुगा गिट्टीको निकासीको लागि IEE आफै गर्ने /अनि ठेक्का लागउने प्राबधानमा सुधार हुनुपर्ने
- दक्ष जनशक्ति को अभाव,देश भर 2दर्जन भन्दा कम स्थायी वातवारण निरिक्षक
- कुनै परियोजनाको वातावरणीय प्रतिवेदन राजनीतिक दवावले रातारात हुने गरेको अवस्थामा सुधार हुनुपर्ने जस्तै पर्साको प्रस्तावित चिडियाखाना
- EMP का प्राबधानहरु अनिवार्य लागू गराउनुपर्ने सुनिश्चितता हुनुपर्ने
- EnvironmentalMonitoring मा बन तथा वातावरणको अधिकार नै नहुनु ,नतिजामूलक अनुगमन नहुनु,अनुगमनको नाममा कर्मकाण्डी/तीर्थाटन हुने अवस्थामा सुधार हुनुपर्ने
- जरिवानाको प्राबधानले पैसा तिरेर वातवारणीय अध्ययन ढिला गर्दा /वा पेलेर जान सकिने मनसाय जस्तै चन्द्रागिरी रिसोर्ट काठमाण्डौ मा सुधार हुनुपर्ने आदि

Water for irrigation is being treated in isolation some efforts have been made in the past to integrate irrigation with power generation, please discuss your concept on IWRMP with special reference to hill irrigation/Tar irrigation in Nepal. - रा.प.प्रथम

उत्तर : पृष्ठभूमि :

नेपालको सिंचाइ क्षेत्रको विकास क्षेत्रगत तरीकाले हुदै आएको छ ।

- सिंचाइ नीति
- सिंचाइ नियमावली
- सिंचाइ मन्त्रालय/विभाग - जलश्रोत तथा सिंचाइ मन्त्रालय/विभाग

सिंचाइ सम्भाव्यताको आधारमा योजना तर्जुमा हुने, कार्यान्वयन हुने, अन्य क्षेत्रको/ निकायको आवश्यकतालाई ध्यान नदिने, पानी प्रशस्त र जनसंख्या कम हुदा यस प्रकृतिका विकासका नकारात्मक असर नदेखिएपनि हाल जनसंख्या वृद्दि भैरहेको, स्वच्छ पानीमा भास्पधा भैरहेको. पर्यावरणीय र वातावरणीयस्वच्छता सम्बन्धी जनचेतना अभिवडी भैरहेकोले समस्या देखिन थालेको छ ।

मुलभूत समस्या :

एकीकृत जलश्रोत व्यवस्थापनको सिद्दान्त (IWRM) को आधारमा सिंचाइ क्षेत्रको विकास र विस्तार नहुनु

समस्याका कारणहरु:

- जलश्रोत रणनीति (WRS) र राष्ट्रिय जलयोजना (NWP) ले एकीकृत जलश्रोत व्यवस्थापनको सिद्धान्तको आधारमा जलश्रोतको विकास र व्यवस्थापन गर्न रणनीति र साजना तर्जुमा गरेतापनि सो अनरुप जलश्रोत ऐन र एकीकृत जलश्रोत नीति नभएको ।
- सिंचाइ विभागको संगठनात्मक व्यवस्था नदि वेसिन्मा आधारित नभएको
- जलश्रोत सम्बन्धि समन्वय. विवाद व्यवस्थापन, जलश्रोत नियमन अनुगमन गर्ने निकायको अभाव
- वेसिन तहमा तथ्यांकको अभाव,
  - जनसंख्या,
  - विभिन्न उपयोगको लागि जलश्रोतको परिमाणात्मक आवश्यकता,
  - उपलब्ध जल परिमाण,
  - जमीन सम्बन्धी तथ्यांक,
  - Watershed को आवश्यकता,
  - वातावरणीय र पर्यावरणीय आवश्यकता,
  - महिला र विभिन्न सिमान्त वर्गको अवस्था,
- क्षेत्रगत उद्देश्यले उपभोक्ता समिति गठन भएतापनि बेसिन तहमा समिति गठन नभएको
- उर्जा र सिंचाइ दुइ छुट्टाछुट्टै क्षेत्रगत उपयोगका लागि छुट्टाछुट्टै विभाग भएकोले बहुउद्देश्यीय योजनाको नेतृत्व लिने निकायको अस्पष्टता

समस्याको प्रभाव : -

- जलश्रोत ऐनले जलश्रोत उपयोगमा प्राथमिकताक्रम Rigid बनाएकोले सिंचाइमा प्रयोग हुने पानीको अधिकार कायम गर्न कठिनाई,
- निजि क्षेत्रको लगानी आउन नसकेको,
- विभिन्न क्षेत्रगत विवाद, सिंचाइ-विधुत, सिंचाइ-खानेपानी
- U/S र D/S विवाद -आाधी खोला, पश्चिम राप्ति, प्रगन्गा र बदकापथ
- सिंचाइ क्षेत्रले अधिकतम फाइदा दिन नसकेको,
- सुख्खा समयमा वातावरणीय र पर्यवारणीय आवश्यकताको लागि मा पानी नछोडिनु,
- सिंचाइ योजनाहरु बाढी, भू-क्षय, Excessive sediment जस्ता समस्याहरुबाट ग्रसित रहनु,

समस्या समाधानको लागि वर्तमान नीति तथा कार्यक्रम

(i) जलश्रोत एन र नियमावली :

उपभोक्ता समिति गठन विवाद समाधान संयन्त्र, अनुमति लिने व्यवस्था, वातावरणीय संरक्षण, जल प्रदुषण रोक्न कुराहरु जलश्रोत ऐनले नेपाल राज्य मातहत रहने व्यवस्था गरेको विषय आदि ।

(ii) जलश्रोत रणनीति र राष्ट्रिय जल योजना :

एकीकृत जलश्रोत व्यवस्थापनको सिद्दान्त (IWRM) मा आधारित रहेर जलश्रोतको विकास र व्यवस्थापन हुने गरी तयार भएको ।

- IWRM सिद्दान्त अनुरुप ऐन र नीति.
- संगठनात्मक व्यवस्था,
- जलश्रोत हेर्ने केन्द्रिय निकायको व्यवस्था.
- बेसिन कार्यालय, वेसिन तहमा उपभोक्ता समिति, जिल्ला जल सभा,
- DWRC लाई पुनर्गठित गरी श्रोतसाधन र अधिकार सम्पन्न पार्ने विषय,

(ii) सिंचाइ नीति : .

- IWRM सिद्दान्त अपनाउने
- वेसिन तहमा गुरु योजना तयारी
- स्थानीय निकाय र ज.उ.स. को योजना तयारी देखिने सक्रिय सहभागिता
- वातावरणीय सरोकार समेटिएको,
- महिला सशक्तिकरण, लैंगिकसमानता र पिछडिएको क्षेत्रमा विशेष सहुलियतसहितको सिंचाइ कार्यक्रम
- हाल सम्मका पहाडी क्षेत्रमा सिंचाइ सुविधा समेत पुग्ने गरी बनाइएका बहुउद्देशीय योजनाहरु
- पहाडका नदीहरु गहिरिएर बग्ने हुनाले टारमा सिंचाइ आवश्यक पर्ने, त्यसैले पहाडी क्षेत्रमा Snow fed River बाट सिंचाइ सुविधा कम मात्र भएको,
- धेरै जसो प्रणालीमा साना र मझौला स्तरका खोला नदीहरुको पानी प्रयोग भएको,
- केहि स्थानमा बहउद्देशीय समेत बनेको :

आरुटार -गोरखा

चौर जहारी - रुकुम

पोखरा जलउपयोग - कास्की

आँधीखोला - स्याङ्जा आदि

त्यसैले सिंचाइ मात्र प्रयोजनको लागि सम्भाव्य नभए पनि बहुउद्देश्यीय योजना बनाएर हाडी टारहरुमा सिंचाइ सविधा विस्तार गर्ने सम्भावना रहेको छ ।

सुझावहरुः

१) IWRM सिद्दान्त अपनाउने

- एकीकृत जलस्रोत नीति
- जलश्रोत ऐन संशोधन
- विभिन्न तथ्यांकहरुको संकलन
- वेसिनमा आधारित गुरुयोजना
- समाजमा कमजोरवर्गलाई महशल सुविधा दिएपनि, जलश्रोतलाई आर्थिक वस्त (Economical Goods) मानेर लागत उठतीको सिद्दान्त,
- वातावरणीय र पर्यावरणीय पक्षमा ध्यान दिने,
- सरोकारवालाहरुको सहभागिता. समन्वय, सहमति र स्वामित्वलाई योजना तर्जुमा अधिकार,
- सामाजिक अवस्थाको अध्ययन. आवश्यकता र ग्रहण गर्न सक्ने क्षमताको विश्लेषण
- बेसिनमा आधारित प्रशासनिक संगठनात्मक र ज.उ.स.को गठन व्यवस्था

२) **पहाडी तारमा सिंचाइ योजना विस्तार गर्ने**

- बेसिनको गुरु योजना तयारी,
- सिंचाइलाई सकेसम्म बहउद्देश्यीय बनाउने,
- घरायसी उपयोग,
- विधुत,
- सिंचाइ,
- -विधतबापत प्राप्त आम्दानी सिंचाइ व्यवस्था दिगो बनाउन र लागत उठाउन प्रयोग गर्ने,
- सिंचाइ प्रणाली मर्मत सुधार र संचालनमा पर्याप्त हुने गरि ISF उठाउने,
- कृषकलाई पर्याप्त लाभ हुने गरि कृषि व्यवसायिक वजारको व्यवस्था आदि,
- समाजमा कमजोर वर्गलाई सहुलियत तर यस्तो सहुलियत सम्बन्धी निर्णय प्रक्रियामा पारदर्शिता

यसरी पहाडी टारहरुमा उपयुक्त हावापानी र माटो जमीनको भिरालोपन Moderate खालको हुने भएकोले उब्जाउको र बाली विविधिकरणको उच्च सम्भाव्य रहेको छ।

यसरीजल र जमीन आदी श्रोतको अधिकतम उपयोग गरी समाजका जल सम्बन्धी अन्य आवश्यकता समेत पुरा हुने र वातावरणीय पक्षलाई ध्यान दिई योजना गर्न सकिएमा आर्थिक सामाजिक अवस्था थोरै समयमै परिवर्तन ल्याउन सकिन्छ।

सिंचाइको प्रविधि र लागत :पहाड र उच्च पहाडमा

- Soil (Course Textured, पातलो) को कारणले Seepage.loss बढी हुने भएकोले Lining गर्नु पर्ने
- अधिकांश नहर Contour Canal बनाउनु पर्ने,
- Contour Canal मा धेरै Cross Drainage Structures र पहिरो रोकथामका उपायहरु अवलम्बन गर्नुपर्ने,
- Slopy Topography भएकोले Steep Canal बनाउनु पर्ने र Erosion रोक्न Lining गर्नुपर्ने,
- River Toe Cutting रोक्न संरचनाहरु बनाउनु पर्ने,
- साना Command Area तर तुलनात्मक रुपमा लामा लामो Ideal length भएका Long Canal बनाउनु पर्ने
- नदीबाट पानी Diversion गर्न र Debris Deposition को कारणले Sub Surface Flow रोक्न Bed Seal बनाउनु पर्ने,
- Soil Topography on corum Conveyance Efficiency and Water application efficiency कम हुने भएकोले बढी म्गतथ हुने गरेको
- दुर्गम भू-भाग र निर्माण सामग्रीको लागत महँगो पर्ने,

यी सबै कारणले तराइको तुलनामा पहाड र उच्च पहाडमा सिंचाइ प्रणाली निर्माणको लागत बढी पर्ने

कृषिको सम्भावना :

- बाली विविधिकरण र उच्च मूल्यका खेतीको प्रचुर सम्भावना
- पहाडी टारहरुमा धान र अन्य खेती (तरकारी) तराईको तुलनामा बढी उत्पादकत्व हुने,
- SlopyTerrain मा मकै, आलु,जडिबुटी,फलफुल र डाले घाँसको खेती गर्न सकिने,
- Organic खेतीको बढी सम्भावना,
- रोग व्याधि, किरा, फट्यांग्राको प्रकोप कम,

त्यसैले पहाडी र उच्च पहाडी क्षेत्रमा सिंचाइ विस्तारको औचित्य र पष्ट्याईः

१) देशमा उपलब्ध प्राकृतिक श्रोत जल र जमीनको उचित उपयोग गरी अधिकतम फाइदा लिन सकिने,

२) कषि व्यवसायीकरण र विविधिकरण गर्ने,

३) उच्च मल्यका खेती गरी निकासी व्यापारबाट कृषि व्यापार घाटा कम गर्न

४) देशमा सन्तुलित विकास, क्षेत्रीय सन्तुलन कायम गर्न,
५) पहाडबाट तराइमा हुने बसाई सराइ रोकी जनसंख्या व्यवस्थापन गर्न,
६) पहाडी क्षेत्रमा गरिवी निवारण गर्न र आर्थिक क्रियाकलाप बृद्धीगर्न
७) खाद्य सुरक्षा बढाउन
८) कतिपय पहाडी सिंचाइमा Micro ज्थमचय को समेत विकास गरिएको

Ground Water Tubewell Cluster Model मा विकास गर्दाका फाइदाहरु

- प्रभावकारिता बढ्ने,
- अन्य पूर्वाधार सडक, विधतीकरण पनि संग संगै विकास हुने,
- कृषि प्रसार प्रविधि पुर्याउन सजिलो,
- कृषिको Priority Pocket क्षेत्रको रुपमा विकास गर्न सकिने,
- आयोजना कार्यान्वयनको लागि छुट्टै आयोजना कार्यालय,
- योजनाको लागत प्रति हेक्टर कम गर्न,
- समग्र सिंचित क्षेत्रको कृषिमा सकारात्मक प्रभाव पार्ने,

Explain how Water resources management planning. opportunities, threats organization actuation and controlling can be done.

१) नीतिगत :
- IWRM को नीति कार्यान्वयन नहुनु,
- Basin कार्यालय स्थापना नहुनु,
- जलश्रोत ऐन/नियमावली समयसापेक्ष अध्यावधिक नगरिनु,
- अन्य क्षेत्रगत ऐन र जलश्रोत ऐनको तादाम्यताको (Coordination) कमी,
- स्थानीय निकायको क्षमता सदढीकरण र अधिकार निक्षेपण नहन, - एकीकृत जलश्रोत नीतिको अभाव,
- Resettlement Policy को अभाव,
- Integrated basin management नहुनु,
- Research and development को कमी,
- GW policy नहुनु,
- GW को अनुगमन, नियन्त्रण,नियमन, संरक्षणको लागि कानुनी संयन्त्रको अभाव,

२) संस्थागत
- Fragmented , Sectoral Utilization मा focus भएको,
- समग्र जलश्रोत व्यवस्थापन, संरक्षण र नियमन गर्ने निकायको अभाव.
- समन्वय र नियमनगर्ने निकायको अभाव,
- भएका संस्थाहरु पनि क्षेत्रगत उपयोगको अवधारणाबाट टुक्र्याउन,
- भुमिगत जलश्रोत व्यवस्थापन गर्ने निकायको अस्पष्टता,
- Hydrological Basin को आधारमा कार्यालय स्थापना नहुनु,

३) वातावरणीय
- वातावरणीय कानुनको प्रभावकारी कार्यान्वयन नहुनु,
- EIA/IEE /EMP को अनुगमन,
- Water Body मा मिसाइने Effluent Standard र नियमनको अभाव,
    - जल प्रदुषणका सीमा नतोकिनु,
    - सुख्खायाममा Environmental flow को लागि पानी नछोडिन.

४) **Water Induced Disaster - WID**
- अत्याधिक River Manning
- Landslide and Soil Erosion

- Debris flow and GLOF

५) राजनीतिक

- अस्थिरता
- नीतिगत प्रतिवद्दता
- संविधान संरचनामा जलश्रोतको बाडफाँड
- चुहावट, भ्रष्टाचार

७) Climatechange (जलवायु परिवर्तन)

८) Law use efficiency

९) सेवा शुल्क नउठ्नु -Further development को लागि वजेटको अभाव

१०) सेवा प्रवाहमा समानता

११) Land Use Policy को अभाव

Process and stages of IMTPM

(A) Stages of IMTP

1) Initial organization
2) Management Transfer Preparation
3) Management Transfer Implementation
4) Post – Turnover Activities

(1) Initial organization

- Establishment of sub–project office and deputation of Staff
- Introductory study
- Base line study
- Household and irrigation area Survey
- WUA formation and registration
- Establishment of sub–project-Management committee
- WUA office Establishment
- WUA strengthening activities by formal and informal training.

(2) Management Transfer Preparation

- Joint Diagnostic walk – through
- Identification of rehabilitation and Improvement works
- Prioritization of identification needs
- Cost sharing arrangement between GON and WUA
- Preparation of resource mobilization plan
- Preparation of draft action plan and draft memorandum of agreement
- Discussion and Finalization of AP and MOA
- Singing MOA for implementation of AP
- Training program continued

3) Management Transfer Implementation

- Training program to strengthen WUA capability
- Approval of design and estimate, bid documentation and preparation and bid Invitation.
- Construction Management and Quality Control
- Preparation Management of O&M Plan &M Plan & Expenditure Plan (ISF fixation)

- Water Management Activities
- Monitoring, Evaluation & Feedback
- Formal Management Transfer Program Execution.

4) Post–Transfer Activities

- Continuation of training program to strengthen WUA Capability
- Regular O & M of canal by WUA
- Water management activities continued .
- Development of WUA / Agricultural support services relationship
- Monitoring, Evaluation and Feedback.

Preconditions of IMTP

a) In the beginning, GON ieDOI is expected to assume work as partner with WUA until it becomes capable to assume full responsibilities.

b) The WUA are expected to evolve as democratic institution with equal opportunity for every potential member in the WUA.

c) Functional irrigation facilities are essential conditions for successful IMTP efforts.

d) Additional important aspect of management transfer is improvement agricultural support services to improve the performance of irrigated agriculture.

B) Process of IMTP / Activities of IMTP

1) Rehabilitation process
2) O & M process
3) Institutional and financial process
4) Post transfer support

1) Rehabilitation Process:

The government's in the system rehabilitation has often been used as a major incentive for the users to motivate them towards assuming the responsibility of taking over the system for it's management. physical system rehabilitation has also the objectives of improving the effectiveness and sustainability of the irrigation system.

Rehabilitation of physical structures has been tied to institutional progress and this is found to be a useful procedure as these components support one another. The process of rehabilitation brings together the user farmers and government officials and this helps to identify and address the local needs. Transparency of program, budget and expenditures should be emphasized.

2) O&M process:

- Quality trainings on O & M for WUA officials are necessary to carry out o & M activities efficiently and for this realistic O & M budget are important .
- WUA need to know how much it will cost to property operate and maintain their irrigation system.
- The government staff need to work closely with the WUAs to help them develop their own O & M budgets and then based on their irrigation service
- fee (ISF) rate on meeting those O & M costs.Volunteer services may not sustain O&M over the long run.
- ISF and other fee collections need to be rational and based on actual O&M cost to ensure system sustainability.The WUAs need assistance in O&M record keeping and water management.

3) Institutional & financial management process

- The WUA organizational design and structure need to be fairly simple and practical considering time, funding and personal. Women involvement in all IMTP activities will make them to participate fully and effectively.
- WUA most have human and monetaryresources to successful manage their irrigation system.
  The WUAs should be allowed to be paid for they do beyond their shares.Adequate fee collection, disbursements, financial records and accounting are required.
- Long term WUA financial viability and self–sufficiency needs to be stressed.

4) Post transfer support process:

The Post transfer program should include the agricultural support and on farm water management programs. System O & M plan, parcellary maps. etc should also be developed during IMTP implementation stage & should be contained during post transfer period.

Many completed irrigation projects are in the process of being handing over to farmers for efficient water management. Operation and maintenance by direct involvement of beneficianies farmers. Discuss the ideas there in and also the problems………15

उत्तरः

सरकारद्वारा निर्माण र व्यवस्थापन हुदै आएका सिंचाइ प्रणालीहरु उपभोक्ता किशाखारुलाई व्यवस्थापन हस्तान्तरणका कार्यक्रम ९० को दशक बाटै शुरु भएको हो । हाल पनि यो कार्यक्रम चालु छ । सिंचाइ सम्बन्धी सबै नीति, जलश्रोत ऐन, नियमावली, सिंचाइ नीति नियमावली, APP, WRS, NWP, सबैले यसलाई प्राथमिकतामा राखेका छन् ।

यसका प्रक्रियाहरुः

१) कृषक उपभोत्ता समितिको सस्थागत क्षमता सुद्धिढीकरण गर्ने,
२) संरचनाहरुको पुरुत्थान गर्ने,
३) सम्झौता हस्तान्तरणका शर्तहरु तोकेर सम्झौता गरी हस्तान्तरण गर्ने,
४) सिंचाइ प्रणालीको प्रभावकारिताको अनुगमन र मूल्यांकन जलश्रोत तथा सिंचाइ विभाग गर्ने,

यी मुलभुत प्रक्रियामा विभिन्न क्रियाकलापहरु समय परिमार्जन र अध्यावधिक हुदै यो कार्यक्रमलाई जलश्रोत तथा सिंचाइ विभागले निरन्तर अगाडी बढाएको छ । पछिल्लो पटक IWRMP बाट कन्काई, सुनसरी मोंरग, नारायणी र महाकाली सिंचाइ प्रणालीमा लागु गर्दा निम्न अनुसारको प्रक्रिया/क्रियाकलाप बमोजिम कार्यान्वयन गरेको छ ।

(क) सूचना संकलन
- कृषक समुदाय संग परामर्श
- Rapid Appraisal
- Asset Inventory Survey
- Bench Marking Survey
- उपभोक्ता सुची
- Parcellary Map

(ख) व्यवस्थापन हस्तान्तरण कार्यक्रमको तयारीः
- (i) सिंचाइ विभाग र ज. उ. स. बीच अन्तरक्रिया
- (ii) DOI and WUA बीच सैद्धान्तिक सहमति
- (iii) ज.उ.स.को सस्थागत सुदृढीकरण
- (iv) ज.उ.स. र Field Staff को क्षमता विकास
- (v) Essential structure improvement को प्राथमिकीकरण ज.उ.स.को परामर्शमा
- (vi) सिंचाइ प्रणाली सुधार कार्यहरु
- (vii) विभिन्न तहमा Flow Measurement

(ग) मर्मत सुधारका कार्यहरु DOI and WUA बाट कार्यान्वयन :
- (i) नहर संचालन र मर्मत सुधार हाते पुस्तिकाको तयारी

(ii) मर्मत सुधार र संचालनको जिम्मेवारी हस्तान्तरण

(घ) अनुगमन मुल्यांकन र कृषि सम्बन्धि क्रियाकलापहरु :

(i) प्रणालीको Functionality को अनुगमन

(ii) एकीकृत बालि तथा जल व्यवस्थापन मुल्यांकन

(iii) ज.उ.स.को कार्य सम्पादन मुल्यांकन

(iv) नहर संचालनको पुनरावलोकन र सुधार

Ideas and Thoughts about Management Transfer of Irrigation System

१) सरकार व्यवस्थित सिंचाइ प्रणालीको प्राथमिकता र कार्य सम्पादन स्तर कमजोरः

- Irrigation Intensity कम हुने
- Service Delivery मा A. R. P. E. (Adequacy, Reliability, Predictability, Equality) नभएको
- ISF नउठेको,
- मर्मत सम्भार यथोचित हुन नसकेको,
- Design Life Function नगरेको,
- मर्मत सम्भारमा बजेटको कमी, भएकोमा पनि प्रशासनिक खर्च बढी

२) FMIS को राम्रो व्यवस्थापन- सहभागिता, पारदर्शिता, समानता, जवाफदेहित;

३) सरकारी वजेट नया क्षेत्रमा सिंचाइ विस्तारमा केन्द्रित हुनु पर्ने,

४) आर्थिक उदारीकरणको नीति,

५) कृषकलाई व्यवसायीकरणको नीति,

६) प्रभावकारी र किफायती सिंचाइ व्यवस्थापन/सिंचाइ व्यवस्थापनको मुल्य आम्दानीको तुलनामा घट्दै जाने

सुधारका उपायहरु :

१) सरकारको O&M खर्चमा कमी,

२) श्रोत संकलन - ISF बढी,

३) सिंचाइ सेवाको गुणस्तरमा सुधार,

४) सिंचाइ पूर्वाधारको दिगो मर्मत सुधार,

५) बढी कृषि उत्पादन,

६) प्रति इकाई पानीबाट बढी आम्दानी,

७) सेवा प्रवाहमा प्रभावकारिता र समानता बढ्ने,

८) कृषकहरुको आत्म विश्वास अभिवृदी गर्ने,

९) प्रभावकारी र किफायती सिंचाइ व्यवस्थापन,

समस्याहरुः

१) योजना डिजाइन र निर्माण

- प्रणालीमा पानी Acquisition गर्न कठिनाइ (पश्चिमी गण्डक),
- वालुवा फाल्ने संरचना नहुनु (बागमती सिंचाइ आयोजना),
- अत्याधिक नियन्त्रण र संचालनका समस्याहरु,
- फिल्ड स्तरसम्म नहर र संरचना नबन्नु,
- संरचनाहरु जटिल,

२) प्राकृतिक

- अत्याधिक FlowVariation.
- Sediment LadenWater
- डुवान र नदी कटान,

३) ज.उ.स.को संस्थागत क्षमता

- कमजोर संस्थागत क्षमता,
- राजनीतिक दवाब,

- अभिलेख तथाआफ्नै कार्यालय नहुनु,
- श्रोत परिचालनमा समस्या,
- प्राविधिक र व्यवस्थापकीय दक्षतामा कमी,

४) कृषि

- व्यवसायीकरण, विविधिकरण हुन नसक्नु,
- बजार,मुलय र सयन्त्रको अभाव
- agricultural input को उपलब्धतामा कमी हुनु,
- श्रम शक्तिको अभाव जलवायु परिवर्तनको असर

५) जमीन

- खण्डीकरण
- स-साना प्लोट
- गैर कृषि जमीनको बढदो प्रयोग
- Absentee Land Lord

६) कानुनी र नीतिगत

- ISF उठाउन कानुनी लचकता

७) राजनीति र सामाजिक

- राजनीति दवाब,
- पानी आर्थिक मुल्य मान्यता नदिनु ,

GW विस्तार गर्न अपनाइएको रणनीति :

- सम्भाव्य स्थानमा STW विस्तार गर्ने, STW सम्भाव्य नभएको स्थानमा मात्र DTW बनाउने.
- STW Cluster Model (छ गोटा) मा विकास गर्ने,
- Group Ownership मा STW रहने,
- सडक र विधुतीकरण भएको,
- कृषि प्रविधिको विस्तार गर्ने,
- साना कृषकलाई लक्षित गरी सानो हाते पम्प, ढिकी पम्पको विस्तार गर्ने
- एकीकृत वाली तथा जल व्यवस्थापन,
- नयाँ प्रविधिमा आधारित सिंचाइ प्रणालीको विस्तार गर्ने,
- कृषकहरुबाट दुई बाली लगाउने प्रतिवद्दता,
- कृषि कार्यालयबाट Priority Package कार्यक्रम,
- I/NGO बाट सामाजिक विकास कार्यक्रम,
- Conjunctive Use मा जाने,
- Utilization hour बढाउने,
- WRS/NWP को सिफारिश मुताविक भूमिगत जलश्रोत विकास समितिलाई नियमनकारी निकाय बनाउने, GW सिंचाइको कार्यान्वयन जलश्रोत तथा सिंचाइ विभागले गर्ने,
- DTW को मर्मत सम्भार र संचालनको जिम्मेवारी ज.उ.स.लाई हस्तान्तरण गर्ने
- गरिबी बढी भएको क्षेत्रमा कार्यान्वयनमा प्राथमिकता दिने,
- Utilization hour बढाउन विधुत महशुलमा सहुलियत दिने ।

भूमिगत जलश्रोत विस्तारका समस्याहरु :

- बढी संचालन खर्च,
- साना खण्डीकरण जग्गा,
- कृषिको अपर्याप्त प्रचार प्रसार र विस्तार,
- कषिको व्यवसायीकरण नहुनु.
- ग्रामीण विधुतीकरण नहुनु,
- कृषि सडकको समस्या, सस्थागत जिम्मेवारीको अस्पष्टता.
- सस्तो प्रविधिको विकास नहुनु.
- Poor Well Design

Large assets are created in the irrigation sector by investing huge capital cost. What measures DOI has to take to manage, maintain and sustai these assets. Elaborate the details of such asset management plan. रा.प.प्रथम २०७०/९/१९

उत्तर : नेपालमा योजनावाद्द विकासको थालनी भएपछी राज्यले सिंचाइ क्षेत्रमा प्राथमिकता दिएको र आ.व.२०७५/० आ.व.२०७५/०७६ को अन्त्य सम्ममा वर्तमान मूल्यमा करिव 417 गानी भएको छ । यसबाट असंख्य Hydraulic संरचनाहरु, सिंचाइ विकासका धारहरु निर्माण भएको छ । आ.व.२०७५/०७६ सम्म करीब १४,८३,१५० हे. जमीनमा सिंचाइ पूर्वाधार विकास भएको छ । यसमध्ये ३२ गोटा सिंचाइ प्रणालीहरुका ३.२५,९१९ हेक्टर कमाण्ड एरिया संयुक्त व्यवस्थापनमा रहेको छ भने बाकी कृषक समदायद्वारा व्यवस्थापन हुदै आएका छन् । (सिंचाइ वार्षिक पुस्तिका २०७५/७६) ।

सिंचाइको लक्ष्य र उद्देश्य पुरा हुन् यी संरचनाहरूको सम्बन्धमा आधारभूत मान्यतानिम्नानुसार रहेका छन् : -

- Design Life सम्म Fully Functionable हुनु पर्ने,
- Operation and Maintenance (O&M) उपयुक्त तरिकाले समयमा हुनु पर्ने
- O&M को लागि श्रोत साधनको व्यवस्था हुनु पर्ने, श्रोत साधन संकलन गर्ने आधार निश्चित बनाइनुपर्ने
- O&M को जिम्मेवार स्पष्ट निर्क्योल हुनुपर्ने
- O&M को लागि आवश्यक लागत निर्धारण हुनुपर्ने
- संरचनाबाट अपेक्षित फाइदा लिई क्रिशाखारुले कृषि व्यवसायबाट प्रशस्त मुनाफा लिनु पर्ने
- संरचनाहरुको सेवा प्रवाह Adequately , reliably , predictably and equity हुने गरि दुरुस्त हुनुपर्ने
- समग्र सिंचाइ व्यवस्थापन चुस्त र किफायती हुनुपर्ने

यी आधारभूत मान्यताहरु Asset Management Plan (AMP) बनाइ पुरा गर्न सकिन्छ ।

irrigation Asset Management are explained by a number expert, one of which is as follows: "An integrated approach to improving the ability of an irrigation system to deliver at a definite level of service in the most cost–effective manner."

Asset Management Plan (AMP) helps irrigation authorities in the following ways:

(i) Utilize and maintain the condition of its assets at the best possible way and be kept running at a good operating standard,

(ii) Provide a level of service that is consistent with cost–effectiveness and sustainability objectives and

(iii) Improve the system performance. (the broad goals of which are to achieve improved irrigation efficiency and better crop yields less canal damage from uncontrolled water levels, more efficient labor, improved social harmony and improved environment as a result of less diversion or better–quality return flows.)

नेपालमा सिंचाइ क्षेत्रको हालको समस्या :

१) **O&M को कमजोर अवस्था :**

- अपर्याप्त बजेट

- सरकारी बजेट Adhoc Basis मा विनियोजन
- ISF उठन नसकेको, ISF को एकाइ र आधार तोक्न कठिनाइ
- संयुक्त व्यवस्थापनको योजनामा O&M को जिम्मेवारी स्पष्ट हुन नसकेको,

२) संरचनाहरु बिग्रदै गइ अपेक्षित सेवा प्रवाह गर्न नसकेको,

- Water conveyance , acquisition
- Water control
- Inspection Road बिग्रदै जानु,
- Command Area Protection नहुनु,

३) Essential Structure को प्राथमिकता तोक्न कठिनाइ,
४) चाडै Repair and Rehabilitation गर्नुपर्ने,
५) समग्र Irrigated Agriculture बाट राष्ट्र र कृषकहरुले फाइदा लिन नसकेको समस्या समाधानको लागि गर्नु पर्ने उपायहरु :

१) सबै सिंचाइ प्रणालीहरुको Asset Management Plan (AMP) तयार गर्ने,
२) मझौला सिंचाइ योजनाहरु AMP तयार गरी कृषकहरुलाई हस्तान्तरण गर्ने,
३) वृहत र ठूला योजनाहरुको AMP तयार गरि उपभोक्ता समितिको क्षमता सुदृढीकरण गरी क्रमशः हस्तान्तरण गर्ने ।

नीतिगत व्यवस्था

- सिंचाइ नीति २०७० मा AMP तयार की नीति
- बृहत सिंचाइ योजनाको मूल नहर र H//W सरकारबाट व्यवस्थापन, शाखा र प्रशाखा नहर ज.उ.स.को सहभागितामा संयूक्त व्यवस्थापन, प्रशाखा भन्दा मुनि कृषक व्यवस्थापनमा संचालन गर्ने नीति
- मझौला सिंचाइ योजना कृषकहरुले व्यवस्थापन गर्ने, कनै प्राविधिक कठिनाइ भएमा निश्चित समय सम्म सरकारले व्यवस्थापन गर्ने ।

AMP को ढाचा र AMP तयार गर्नेः

1. AssetSurvey:

- Field Survey
- Desk study
- Data base तयारी -H/W, Canal, Control Structure, Road, Buildings, Equipment, drains etc.)
- यी सबै Asset को मुल्य
- Design Life
- मर्मत सम्भारको आवश्यकता (Failure based, Time based, Condition based,Preventative,Corrective)

2. सबै Asset को आकार र Dimension र सूची

Asset को अवस्था, Serviceability (Good, fair,poor, bad) वर्गीकरण गर्ने

3. **Asset opt Function**

- Hydraulic Function
- Operation Function

Function कोवर्गीकरणः

- Fully Functional
- Minor Functional Short Comings
- Seriously Reduced Functionality
- Ceased to Function

4. लगानी क्रियाकलाप, Amount, जिम्मेवारी निर्धारण गर्ने

- आवश्यक श्रोत संकलन र परिचालनको अनुमान,
- ISF Rate र आवश्यक सरकारी बजेटको निक्यौल,
- प्रति इकाई लागत निर्धारण गर्ने,

5. प्राप्त हुन सक्ने आम्दानीको आधारमा लगानी रणनीति निश्चित गर्ने

- लगानीका फाइदाहरु
- रणनीति निश्चित गर्ने

6. संगठनात्मक सुधार

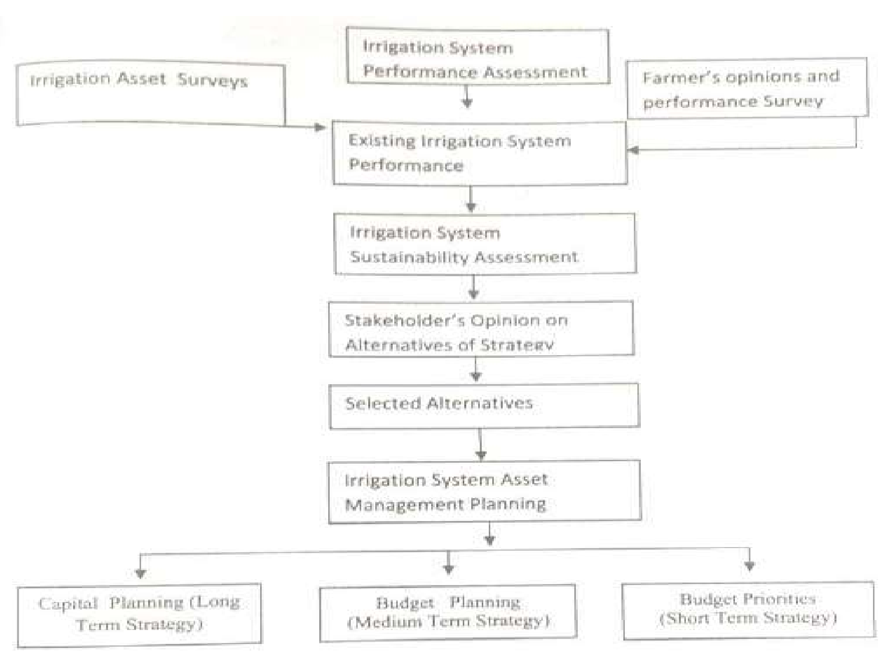
- श्रोत संकलन, परिचालन र जिम्मेवारी निर्वाह गर्न आवश्यक संगठनात्मक परिवर्तन
- तालिम र क्षमता विकासका कार्यक्रम,
- कार्य सम्पादन लक्ष्यको मापदण्ड,

AMP कार्यान्वयन गर्न लागु गर्नुपर्ने:

- Investment आवश्यकतामा आधारित बजेट विनियोजन हुनुपर्ने,
- व्यवस्थापन हस्तान्तरण गरिनु पर्ने,
- कार्य सम्पादन मुल्यांकन,
- स्पष्ट जिम्मेवारी किटान
- जवाफदेहिताको अभिवृद्दी
- Commercial Approach मा लैजाने ।

यी उपायहरु अपनाउन सकिएमा सिंचाइ सेवाका संरचनाहरु Manage, Maintain गरी सेवा प्रवाहलाई दिगो बनाउन सकिन्छ ।

- Method of developing AMP for Irrigation System:
- Assessing system performance is very first stage and major component of an AMP.
- It then followed by appraising the system performance shortfall and it's causes.
- Quantifying the causes of system performance shortfall and seeking the corrective actions needed.
- The last stage is developing an applicable AMP and organizational adjustment needed that enable WUAs managing the system in best–cost effective way,

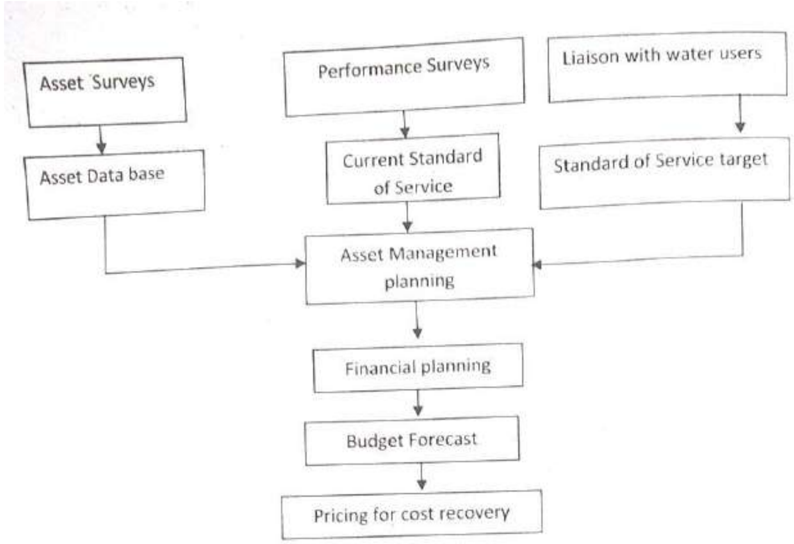


Framework for strategic investment planning of AMP

Framework of AMP –(Proceeding Report 2007 – B.Lohani)

Strategic investment planning based on preparation of AMP focuses on long term anning for investment in irrigation and drainage infrastructure to meet the specific uses defined level of service.

The process involves defining the level of service to be provided, the ability of I uses to pay for that service, the condition of assets (canals, structures access and inspection roads, office etc.) and the investment needed to ain, improve or extend those assets to meet the specified level of service.



Framework for strategic investment planning of AMP
(Source M Burton & perry, 1997)

What are main problems of inundation in the southern border areas of Nepal? Suggest how these problems could be solved, Suggest the monitoring mechanism for solving those problems. - 20

रा.प.प्रथम२०६९/८/७

उत्तरः क) पृष्ठभूमि :

नेपालको तराई क्षेत्रको दक्षिणी सिमावर्ती क्षेत्रमा हरेक वर्ष वर्षादको समयमा डुबान हुने गर्दछ । यस डुबाने स्थानीय जनजीवन, आर्थिक क्रियाकलापलाई नोक्सानी पुर्याउनका साथै राज्य प्रति असन्तोषको भावना समेत बढाउन मद्दत गरेको हुन्छ । यसले गर्दा राज्यले वर्षेनी ठुलो धनजनको क्षति व्यहोर्नु परेको छ ।

- बस्ती र पूर्वाधार डुबानमा पर्ने ।
- खेति र खेतीयोग्य जमिनको नोक्सानी/कटान हुने
- आर्थिक क्रियाकलाप बन्द हुने
- भयावह र आकस्मिक अवस्थाको सिर्जना हुने
- पानी जम्न गइ विभिन्न प्रकारको रोगको प्रकोप फैलिने सम्भावना (Water borne diseases फैलने)

ख) मुलभूत समस्या :

वर्षेनी देशले ठुलो आर्थिक, सामाजिक र मानवीय क्षति व्यहोर्नु परेको ।

ग) समस्याको मुलभूत कारण :

वर्षेनी यस्ता डुवान समस्या हुनुमा निम्नानुसार कारण जिम्मेवार छन् ।

१) प्राकृतिक कारण
२) मानवीय कारण
३) नेपाल- भारत सिमा नजीक बनाइएका संरचनाहरु
४) कोशी र गण्डक डुवानका समस्याहरु

१) प्राकृतिक कारण :

- समथर भूभाग,
- माटोको पानी सोस्ने क्षमताको कमी,
- थोरै समयमा धेरै वर्षा हुनु,
- जलवायु परिवर्तनको असर (Flash flood,high intensity of rainfall e.g. Dang2014, Mahendranagar–2013)
- चुरे क्षेत्रको वन विनाश र Watershed Degradation.
- River Avulsion, Bank cutting, Toe Cutting भइ डुवान हुने,
- Flood Absorption को कमी, (समथर भूभाग र अपर्याप्त Drainage)
- नयाँ प्रर्वत शृंखला र अस्थिर भौगोलिक अवस्थाले गर्दा हिमालय पर्वत शृंखला क्षेत्रका जलाधार क्षेत्रमा बढी मात्रामा भू-क्षय हुने गरेको,

२) **मानवीय कारण :**

१) जनसंख्याको बढ्दो चापले गर्दा बाफ्नो पर्ति एव जंगल कटानी गरि सो ठाउमा कृषिजन्य क्रियाकलाप शुरु भइ प्राकृतिक भू- उपयोग शैलीमा परिवर्तन हुनु,

२) नदीको पानी बग्ने क्षेत्रमा घर टहरा, उधोग व्यवसाय तथा अन्य गतिविधि बढ्दै गएको र दिनानुदिन अतिक्रमण (Encroachment) भइ नदीमा पानी बग्ने बहाव क्षेत्र (Waterway) कम हुदै गई Overbank Spillway हुने समस्या हुने, (तिनाउ नदी -रुपन्देही) ३)

३) नेपालको पुर्व-पश्चिम राजमार्ग र हुलाकी सडकका संरचनाहरुको Drainage Opening नर्म्स र मापदण्ड अनुसार डिजाइन नहुनु,

४) नेपालको शहरी क्षेत्रमा Drainage को अपर्याप्तता, Solid Waste Management को समस्या साथै Stormwater Drainage को समस्या (Drainage Management समस्या नेपालगन्जमा)

३) **नेपाल- भारत सीमा नजीक बनाइएका संरचनाहरु :**

- नेपाल र भारतको सिमावर्ती क्षेत्रमा जमीन सरकारको भएको र भारतमा सीमा नजीक निर्माण हने मुख्य तिन प्रकारका संरचनाहरूले नेपाली भू भाग डुवानमा पर्ने गरेको:

।) भारतमा सिंचाइ गर्ने प्रयोजनको लागि बनाएका संरचनाहरुः

- नदीहरुमा सिंचाइ प्रयोजनको लागि सीमा नजीक बनेको Rarmom गिरीजापुर, लक्ष्मणपुर, वाणगंगा, डण्डाफरेना, कोइलावास आदि,
- सिमावर्ती क्षेत्रमा नेपाली भू भागमा पानी संचय गर्न Damming गरी तलाउ बन्ने र सो पानीभारतमा सिंचाइमा प्रयोग हुनेः महली सागर, बजाहसागर, सिरो सागर, मार्थी सागर -कपिलवस्तु

(ii) भारतमा नदि नियन्त्रण/डवान निराकरणका लागि बनाइएका तटबन्धः

- नेपालको सिमाना नजीक भारतीय बस्ती डुवानबाट बचाउन निर्माण भएका तटबन्धहरु (Bund)
- भारतको वैरगनियामा निर्माण गरिएको च्ष्लन द्यगलम बाट नेपालको गौर बजारमा हुने गरेको डुवान
- भारतको लक्षमणपुरमा राप्ती नदीमा बनाइएको वराज र गाइड वन्ड (२२ कि.मि.) निर्माण तथा नेपालबाट भारतमा बग्दै गरेको गंधेली र सोतीया नालाको प्रवाहलाई पुरै बन्द गरी निर्माण गरिएको कलकलवा बाँध नेपालको बाँके जिल्लाका ८ गा.वि.स.हरु डुवान हुने गरेको,

(iii) भारतमा यातायात र डुवान निराकरण प्रयोजनको लागि बनाइएका संरचनाहरु :

- यातायात र डुवान दुवै उद्देश्य राखी निर्माण भएका Bund हरु Rasiwal - Khurd
- Lotan Bond बाट रुपन्देही जिल्लाको मर्चवार क्षेत्रमा हुने डुवान,
- भरमा निर्माण भएका Railway and Highway/Road सिमानासंग (E–W) बनाइएका संरचनाहरूले प्राकृतिक Sheet flow रोकी डुवान गर्ने गरेको छ ।

४) **कोशी र गण्डकका डुवानका समस्याहरु :-**

- गण्डकको पश्चिमी मुल नहर नेपालबाट जाने र सो को कारणले Drainage congestion, drainage को नियमित मर्मत सुधारको कमीको कारणबाट नवलपरासीका ४ गोटा गा.वि.स.हरु डवानमा पर्ने गरेकोरायपुर,खडौना, पसौंनी आदि ।
- त्यस्तै कोशी पश्चिमी नहरबाट गण्डक प्रकृतिको समस्याको कारणले सप्तरी जिल्लाका दक्षिणी गा.वि.स.हरु डुवानमा पर्ने गरेको ।

घ) **समस्या समाधानको लागि प्रचलित नीति र कार्यक्रम :**

- जल उत्पन्न प्रकोप नियन्त्रण सम्बन्धि कार्यक्रमहरु, WID Management Policy -2072.
- कोशी गण्डक नेपाल भारत संयुक्त समिति-व्क्ष्ँ्ए र अन्य विभिन्न समितिहरु JCIFMJCWR, JSTC, JMCWR.
- ४ गोटा नदीमा डुवान सम्बन्धी Real Time Data को आधारमा Early Warning System.
- भौतिक संरचना डिजाइन को लागि Guidelines and Manuals हरु,
- District Contingency plan - I/NGO and DOI ले बनाउने,

- जनताको तटबन्ध कार्यक्रम,
- चुरे संरक्षण कार्यक्रम.

ङ) डुवान समस्या समाधानको लागि सुझावहरु :

i) जलाधार संरक्षण र व्यवस्थापन - Waterched Conservation and Management
- Soil Erosion, Landslide रोक्ने, (भूक्षय, पहिरो व्यवस्थापन)
- Afforestation मा ध्यान दिने.
- Wet land Conservation - सिमसार क्षेत्रको संरक्षण,

ii) सन्तुलित भू उपयोगमा ध्यान दिने,
- बढी गहिरो स्थानमा बस्ति नबसाल्ने,
- उपयुक्त खेती प्रणाली अपनाउने,
- Land Consolidation गर्ने र Land. Grading Manual (गहिरो खाल्टा खुल्टी पुर्ने),

ii) समुचित इन्जिनियरिंग संरचनाहरुको निर्माण
- शहरी क्षेत्रमा पर्याप्त Drainage को व्यवस्था,
- संरचनाहरुको Opening निर्धारित मापदण्ड अनुसार प्रयाप्त राख्ने,
- StormWater Inlet को प्रावधान राख्ने,

iv) बायो इन्जिनियरिंग प्रविधि अपनाउने तथा वाहब क्षेत्रमा हुने अतिक्रमण न्यनीकरण गर्ने,
- Use of Bio- engineering Technology.
- Control and Mitigation of River Bank Encroachment.
- River avulsion and Toe cutting and Bank cutting रोक्ने,

vi) भारत संग जल कटनीतिलाई प्रभावकारी बनाउने,
- नेपालबाट भारत तर्फ बग्ने नदीनालाहरुको प्राकृतिक वहावलाई रोक्ने गरी तटबन्ध नबनाउने तथा प्राकृतिक रुपमा नै जल प्रवाह गर्न दिने,
- नेपाल भारत सिमानाको समानान्तरमा बनाइएका सडक,राजमार्गमा पर्याप्त मात्रामा निकास संरचना राख्ने व्यवस्था गर्ने,
- नेपाली भू-भाग डुवान हने गरि भारतीय भू-भागमा सिंचाइका लागि संरचनाको एक तर्फी रुपमा निर्माण नगर्ने,
- भारतीय सिमावर्ती क्षेत्रमा बाढी वा सिंचाइका समस्या निराकरण गर्नु पर्ने भए त्यसता निराकरणका उपायको तर्जुमा र कार्यान्वयन गर्नु अगावै दुवै पक्ष बीच छलफल र सहमति पश्चात मात्रै कार्यान्वयन गरिनु पर्ने - Reactive हुनुभन्दा पहिले Proactive हुने,
- सीमा क्षेत्रमा निरन्तर अनुगमन गर्ने कोशी गण्डक सम्झौता अनुरुप कार्य र क्षतिपूर्ति दिने,

च) सुझाव कार्यान्वयनको लागि अनुगमन :
- सुझावलाई क्रियाकलापमा बदल्ने,
- क्रियाकलापहरुको सस्थागत जिम्मेवारी तोक्ने,
- समय सीमा सहितको कार्य योजना तयारी गर्ने,
- श्रोत साधनको व्यवस्था गर्ने,
- कार्य सम्पादन सुचकको निर्धारण गर्ने,

अनुगमन प्रतिवेदनको सुझावहरुलाई सम्बन्धित निकायले पालना गर्ने

Describe Kennedy's silt Theory the head S silt Theory. An irrigation channel issilting badly in reach. State the possible cause for this problem suggesttheremedial measures.

Answer: Kennedy's silt theory was proposed by R.G. Kennedy, an Executive ineer of Punjab P.W. D. He carried out extensive investigations on some of the I reaches in the upper Basic Doab canal system. He selected some straight reaches of the canal section, which had not posed any silting and scouring problems during the previous 30 years or soon.

From the observations and investigation in straight reaches of upper Bari Doab canal system for 30 years Kennedy found that – the silt supporting power in a channel crosssection was mainly dependent upon the generation of the eddies, rising to the surface. These addies are generated due to the friction of the flowing water with the channel surface. The vertical component of these eddies try to move the sediment up, while the weight of the sediment tries to bring it down. That keeping the sediment in suspension. so, if the velocity is sufficient to generate these eddies, so as to keep the sediment just in the suspension silting will be avoided. Based upon this concept, he defined the critical velocity -Vo_ in the channel as the mean velocity which will just keep the channel free from silting and or scoring and he related it to the depth of flow by the equation.

$Vo=c_1 y^{c2}$, where y = depth of flow

When c_1 and c_2 are constant depending upon silt charge and c_1 and c_2 were found to be 0.55 and 0=64 (in MKS)

Therefore, $Vo = 0{=}55*m*y^{0.64}$

Later Kennedy introduced a factor depending upon the type of soil generalize the above equation and named as critical velocity ratio (C.V.R.) & denoted by m, depends upon the grade of silt.

Vo = Critical velocity in the channel in m/s

$=0.55*m*y^{0.64}$

For ponds worse than the standard, the values of on were given form 1.0 to 1.2 and for sands fines than the standards, m was valued between 0.9 to 0.8.

Design Procedure:

1. Assume a trial value of depth y
2. Determine critical velocity by $Vo = 0{=}55*m*y^{o.64}$
   Where m= CVR = v/ V_0
3. Calculate Area (A) = Q/V_0
   Calculate hydraulic radius, from the wetted perimeter and Area assuming 2v: 1H or 1.5 V = 1H side slope Calculate the mean velocity (V) by
   Manning's Formula $V=1/n *R^{2/3}*S^{1/2}$
6. Compare V_o with V

Design Guidelines-WECS

The irrigation canal/channel silting badly in the head reach can hau following reasons of silting of canal.

1. The catchment area of the source river is badly/prone to affected by landelide and deforestation which cause large quantities of wash load (silt and clay) Sand and gravel load.
2. There is lack of provision to minimize silting of irrigation canal i.e. lack of sediment control measures.
3. Presence of large quantity of sediment in the source river causes the ending of silt into the canal.
4. The concept of non– silting and non–scoring velocity in the canal might not takeninto the consideration during the design of canal.
5. The soil type and its particle size analysis might not carried out during the design phase of the canal.

Remedial Measures:

In order to minimize silting of irrigation canals, sediment control measures are used. Which can be classified into preventative measures and conative measures. The preventative measures involved preventing sediment from entering the canal while curative measures involve removing sediment from the main canal.

Preventative measures:

Over the last century, a variety of methods have been developed to exclude sediment from entering into canals and these are as follows

a) Curvature of approach canal
b) Pitched island
c) Guide Vanes
d) Divide wall and stilling pond
e) Tunnel excluders

The tunnel excluder has been found to be one of the most effective means of sediment exclusion.

Curative Measures:

Several methods have developed for ejecting or removing sediment from canals and the more common methods used in Nepal are vortex tube ejectors din Settling basins (Gole&chitale, 1971)
Desander
Flushing pond

नेपालको पहाडी क्षेत्र तथा तराई भूभागमा सिंचाइ विकासको सम्भावना एव चुनौतिहरु उल्लेख गर्नुहोस । पााचौ पञ्चवर्षीय योजनाको थालनीपश्चात सिंचाइ विकासको लागि सरकारी स्तरबाट भएका गरेका प्रयासहरु र सिंचाइ क्षत्रमा भएको सरकारी लगानीको प्रभावकारिताको छोटकरीमा विवेचना गर्नुहोस ।

रा.प.प्रथम इरि २०६७/८/२८

उत्तर : नेपालको पहाडी भूभागमा सिंचाइ विकासको सम्भावना एव चुनौतीहरु (Issues of Hill Irrigation System and ways of addressing these issues on policy level)

नेपालको पहाडी क्षेत्रमा खेती गरिएको जमीन १०,५४,००० हे. मध्ये ३,६८ ००० हे. जमीन सिंचाइ योग्य रहेको छ । अहिले विकास भइ रहेको नया प्रविधिको उपयोग गरेर परम्परागत तरिकाबाट सिंचाइ योग्य नभएको जमिनमा पनि सिंचाइ सुविधा पुर्याउन सकिन्छ । तर हाल सिंचाइ योग्य जमिन मध्ये ५४% जमिनमा मात्र सिंचाइ सुविधा पुगेको छ ।

पहाडी क्षेत्रमा सिंचाइ विकासका मुख्य सवालहरु निम्न अनुसार रहेका छन् :

क) कमजोर भौगोर्भिक अवस्था :

पहाडी क्षेत्रको Geology fragile भएको कारण नहर र संरचना हरुको स्थिरतामा समस्या पर्ने गरेको छ । भूक्षय र पहिरोले गर्दा नहर Alignment Wash इगत गर्ने वा नहरमा Debris थुप्रिएर नहर Block गर्ने जस्ता समस्या बारम्बार देखिने गरेको छ । यसले गर्दा सिंचाइ विकास सम्बन्धी कार्यहरु पहाडी क्षेत्रमा महागो पर्ने गरेको छ ।

ख) Seasonal Water flow:

पहाडी क्षेत्रमा ठूला मझौला नदीहरु गहिरिएर बग्ने र सिंचाइ गर्ने जमिन उच्चा Elevation हुने भएकोले ती नदीको पानी प्रयोग गर्न लिफ्ट प्रणाली अपनाउन पर्ने वा Gravitational Canal बनाउनु पर्ने हुन्छ । त्यसैले स साना खोलाहरु (जुन ठुला नदीका Tributary हुन) को पानी सिंचाइमा प्रयोग गरिएको हुन्छ जुन धेरै Seasonal प्रकृतिका

ग) स-साना छरिएका कमाण्ड क्षेत्र :

पहाडी क्षेत्रमा सिंचाइ योग्य परम्परागत तरिकाले सिंचाइ योग्य नभए पनि खेति गरिएको जमिन धेरै चारिएर रहेका हुन्छन् । ति स-साना कमाण्ड क्षेत्रमा सिंचाइ सुविधा पुर्याउन धेरै लामा Idle length भएका नहरहरुको प्रयोग गर्नु पर्ने हुन्छ । नहरहरू प्राय जसो Contour Canal मा बन्ने भएकोले CD संरचनाहरु समेत धेरै हुन्छन । त्यसैले सिंचा प्रणालीको लागत बढदैछ ।

अन्य सवालहरुः -

- Seepage loss due to high porosity of soil (high percolation losses) In the CA and sleep gradient of the bank

- Higher cost of transportation of construction materials like cement, steal, pipes due to site accessibility in Hills
- Laborscarcity for irrigation and agricultural
- Shifting behavior of rivers at intake site
- Steeply terraced hill slopes being less productive than the valleys and tars i.e. Benefits very according to the soil and terrain
- Slope stabilization and Bio – engineering सम्बन्धी कार्यहरु Sustainability को लागि धेरै बचावट गर्नु पर्दछ ।
- Hill Side erosion and slide problem भएको स्थानमा Covered गर्नु पर्ने हुन्छ । Non–gauged river HI Discharge estimation Tour donnect hydrograph –Region)
- Hills मा १०,५४,००० मध्ये ३,६९,००० हे. र Mountain मा २,२७,००० मध्ये ६०,००० हे मा सिंचाइ सुविधा पुगेको छ ।

तराई क्षेत्रका सवालहरु :

१) **Sediment:**

तराइ क्षेत्रमा सिंचाइ गर्नको लागि Headworks भावर क्षेत्रमा बनाउनु पर्ने जहा चुरिया Erosion को कारणले धेरै Sediment रहेको हुन्छ, Sediment लाई Hydraulically flush गर्न Head को समेत समस्या पर्ने गरेको छ ।

२) **जग्गा प्राप्तिको समस्या** :

सिंचाइ संरचना बनाउनको लागि तराइ क्षेत्रमा जनघनत्व धेरै भएकोले जग्गा प्राप्तिको समस्या देखिने गरेको छ । ३)

३) **Seasonal Water sources**:

चुरिया नदीहरु धेरै ाबिकजथ हुने र मझौला नदीहरु समेतको Discharge variation , हने भएकोले यी नदीहरुबाट वर्षे भरि सिंचाइ सुविधा हुन नसक्ने अवस्था रहेको छ । या नदीहरुमा Storage Facilities को लागि लगानी जुटाउन कठिन र भारत संग सहमति समेत हुनु पर्ने देखिन्छ । । कोशी, गण्डकी, कर्णालीमा मात्र भारत संग सहमति भएको र अन्य His benefit लगायतका विषयमा सहमति बन्न सकेको छैन, क्षेत्रका सिंचाइ विकास सम्बन्धि केहि अन्य चुनौतीहरु :

1. Require huge investment for flood control structures and inundation Management.
2. Comparatively large irrigation system in Terai. So, it required different levels of canals and canal structures as well as distribution outlets.
3. Problem of illegal off takes from canals.
4. Problem of drainage in low land area/inundation in low land
5. Problem of land fragmentation. No use of CA for agriculture
6. केही स्थानमा Water logging and salinization को समस्या हुने
7. Structural protection/stability को लागि Scour protection गर्नु पर्ने
8. धेरै lining कार्यको आवश्यकता नपरे पनि Village and Residential Area बाट गएका नहरमा Lining/ Covered Canal आवश्यक पर्ने
9. Migration को कारणले र धेरै नै absentee land holders भएकोले Participatory irrigation Management को सहमतिमा पुग्न समय लाग्छ ।
10. MIP Method अनुसार एउटै Hydrograph use गर्न सकिन्छ(Hydrological Regio-7)
11. तराईको १३,६००० हे खेतीयोग्य जमिन मध्ये १३,३८,००० हे. जमिनमा सिंचाइ सुविधा पुगेको छ ।

यी सवाललाई नीतिगत स्तरबाट समाधान गर्ने उपायहरु :

सवालहरु	नीतिगत स्तरमा समाधान गर्ने उपाय

क) पहाडी क्षेत्रका सवालहरु १) कमजोर भौगर्भिक अवस्था	- एकीकृत जलाश्रित व्यवस्थापनको सिद्दान्त अपनाउने - भू संरक्षण का कार्यहरु गर्न भू संरक्षण विभाग संग समन्वय गर्ने । - माटो कम काट्ने, धेरै ठाडो बनाएर नकाट्ने - Bio - Engineering लाई प्रवर्धन गर्ने । - योजना तर्जुमा चरणमा नै प्रशस्त Protection work समावेश गर्ने ।
२) Seasonal Water flow -मौसमी उपलब्ध पानी	२) - पानीको उपलब्धता अनुसारको खेति प्रणाली अपनाउनु पर्ने । - उच्च मुल्यको बालीको लागि लिफ्ट अपनाउने - Recycle , Reuse , Reduce and Recovery का सिद्धान्त अपनाउने - Water use efficiency बढाउने
३) स - साना छरीरिएर रहेका कमाण्ड क्षेत्र	३) - नया प्रविधिका .Drip / Sprinker hasvesting pond जस्ता प्रविधिको विकास गर्ने - Water Conservation गर्ने ।
ख) तराइ क्षेत्रका सवालहरु i) Sediment को समस्या	१)- Sand mining लाई नियन्त्रण गर्ने - उपयुक्त संरचना र संचालन तरिकाबाट नियन्त्र गर्ने - Watershed Conservation अपनाउने
ii) जग्गा प्राप्तिको समस्या	२) - योजना शुरु गर्नु भन्दा अगावै जग्गा प्राप्तिको समस्या समाधान गर्ने - मुल्यांकलाई व्यवहारिक बनाउन मुल्यांकन समितिमा स्थानीय व्यक्ति समेतको सहभागिता गराउने - जग्गा प्राप्ति एन २०३४ लाई समय सापेक्ष, रुपमा संशोधन गरि मुआब्जा सम्बन्धि सवालका सम्बोधन गर्ने ।
3) Seasonal water source	3) - भारत संग छलफल गरि सहमतिमा पुग्ने - Storage / Inter Basin water transfer project बनाउने

Water Induced Disaster -WID_ are increasing day by day in Nepal also. The debris and sand flow are affecting the river regimes and constructed infrastructure facilities are in constant vulnerability= State the short, medium and long–term interventions to be adapted for a sustainable development endeavour.–15 रा.प. प्रथम २०६६/९/१६

उत्तर : नेपालमा जाला उत्पन्न प्रकोपका घटनाहरु वर्षेनी बढ्दै गएका छन् । बाढी , पहिरो,ग्रेग्रान वहाव , डुवान Sedimentation समस्याहरु हरेक वर्ष उत्पन्न भइ निर्मित पूर्वाधार संरचनाहरुलाई जोखिम बढाई रहेका छन् ।
Debris andSand flow को कारणले कुलेखानी Reservoir को Dead storage सन् १९९३ मा आधा भरियो । मलेखुको पुल दुइ चोटी Waste यगत भयो । राजमार्ग हरु River Toe cutting को कारणले विग्रिएका छन् र वर्षातमा Debris flow र पहिरोको कारणले अवरुद्द हुनुका साथै मर्मत सम्भार खर्च समेत बढेको छ । सिंचाइ H/W र Intake बाट Sediment Entry भइ Water acquisition र Conveyance मा कठिनाइ भएको छ भने मर्मत

सम्भार र संचालन खच पनि बढेको छ। सन् १९७० मा तिनाउ नदीको पुल बगाउनुका साथै धेरै घर हरु Debris ले पुरेको थियो। सन् १९१४ को मनसुनमा जुरेको पहिरोले २ कि.मि.राजमार्ग १९६ गर्यो र सुनकोशी हाइड्रोपावर को ब्यारेज समेत Non -Functional बनाएको छ।

यी समस्या आउनुमा निम्नानुसारका कारणहरु जिम्मेवार छन् :

क) प्राकृतिक कारण

1) Geology कमजोर, Active Techtronic, fault/fold, cleavage
2) Topography - sleep
3) Climate - high intensity of rainfall, तापक्रममा भिन्नता
4) Hydrology- साघुरो र घुमाउरो नदीहरु
   - River Toe Cutting
   - River degradation .

ख) मानवीय कारण :

- अनुपयुक्त खेति प्रणाली
- बढी भिरालो जमिनमा खेती
- खोरिया फडानी
- वन जंगलको विनाश
- डढेलो
- अनुपयुक्त योजना डिजाइन र तर्जुमा
- योजना तर्जुमामा Sediment flow लाई विचार नगर्नु, तथ्यांकको अभाव
- Watershed degradation
- एकीकृत ढंगले संरक्षणमा प्रयास नहनु
- जनचेतनाको अभाव
- Land use planning and enforcements को अभाव
- Hazard mapping and zoning नहुनु

विकास प्रयासलाई यस्ता समस्याबाट असर पर्न नदिई दिगो बनाउन निम्न अनुसारका कार्यक्रम कार्यान्वयन गर्नुपर्ने

Short-term:

- Watershed संरक्षण Debris flow,Erosion, Land degradation सम्बन्धमा व्यापक शिक्षा र जनचेतना विस्तार गर्ने
- कृषि प्रणालीमा सुधार गर्ने, २५% भन्दा बढी भिरालो जमीनमा खेती नगर्ने, डाले घास र फलफुल खेती गर्ने
- चरिचरणलाइ व्यवस्थित बनाउने
- कमजोर भौगर्भिक बनोट रहेको चुरे संरक्षण गर्ने, गुरु योजना बनाउने
- Hazard Mapping and Zoning गर्ने।
- Maps को आधारमा Land Use Plan तयार गरि कार्यान्वयन गर्ने
- नदि Toe Cutting र Bank erosion रोक्न नदि गुरु योजना बनाउने हरेक Morphology Change हुने भएकोले गुरु योजनामा अपडेट गर्ने
- जथाभावी भारी उपकरण प्रयोग गरेर ग्रामीण सडक निर्माण कार्यलाई रोकी व्यवस्थित बनाउने
- Lower Himalayan Zone र चुरे क्षेत्र कमजोर भएकोले संरचनाहरु उपयुक्त तरिकाले डिजाइन गर्ने
- धेरै Earth Cutting नगर्ने
- जमीनको slope मिलाउने
- Canal short बनाउने र lining गर्ने
- Bridge clear opening (clearance height) धेरै छोड्ने
- Reservoir planning गर्दा Low level मा Flushing राख्ने, Spillway उपयुक्त बनाउने

- चुरे क्षेत्रमा भइ रहेको ढुंगा, गिट्टी बालुवाको उत्खनन रोक्ने
- यी सबै क्रियाकलापको लागि नीति, कानुन, Design, manuals, guidelines तयार गर्ने
- सस्थागत जिम्मेवारी तोकी क्षमता सुदृधीकरण गर्ने
- जिम्मेवारी नदोहोरिने गरी तोक्ने (हाल) GWDB, DSCWSM, DHM दोहोरिने कार्य जिम्मेवारी रहेको

मध्यकाल :

१) Marginal Land क्षतिपूर्ति दिएर राज्यको मातहत लेराउने
२) सिमान्तकृत कृषकलाई वैकल्पिक रोजगारीको व्यस्था गर्ने
३) संरचनाका डिजाइन हरु योजना तयारी, कार्यान्वयन कडाइका पुर्वक अनुगमन गर्ने
४) Land use planning लाइ कडाइका पुर्वक लागु गर्ने,
५) Food plain र नदीका किनारामा Green Belt, मनोरन्जनात्मक पार्क बनाउने.
६) संरचना, सिंचाइ प्रणाली मर्मत सम्भार र संचालनको गाइड लाइन अनुसार सुधारका कार्यहरु लागु गर्ने,
७) चुरे क्षेत्रको संरक्षण को लागि बस्ती हटाइ पुनर्वास कार्यक्रम लागु गर्ने,
८) Bio-Engineering लाइ प्रवर्धन गर्ने,
९) संरचना निर्माण गर्दा वातावरणीय व्यवस्थापन योजनालाई संग संगै कार्यान्वयन गर्ने
१०) Gulley protection को लागि चेक ड्याम निर्माण गर्ने, Drainage को व्यवस्था Catch Drain आदि निर्माण गर्ने

दिर्घकाल :

१) प्राकृतिक श्रोत, जल, जमिन जंगलको संरक्षणलाई दिगो बनाउने
२) नदीमा Sediment Content को निरन्तर अनुगमन र उपचारात्मक/रोकथाम गर्ने
३) पहाडका छरिएका बस्तीहरुलाई एकीकृत गर्ने
४) नदीहरुबाट निश्चित स्थानबाट निश्चित परिमाण को नदिजन्य पदार्थको उस
५) IWRM लागु गर्ने
६) Peak Discharge घटाउन storage project बनाउने, Detention pond बनाउने
७) Watershed Conservation र वन क्षेत्रको व्यवस्थापन सुदृढ बनाउने
८) नीति, नियम, कानुनको निरन्तर सुधार गर्ने,